CHAPTER 9

DESIGN OF LONGITUDINAL STRESS-LAMINATED DECK SUPERSTRUCTURES

9.1 INTRODUCTION

Longitudinal stress-laminated deck superstructures consist of a series of lumber laminations that are placed edgewise between supports and are compressed transversely with high-strength prestressing elements (Figure 9-1). The bridges are similar in configuration to glulam or nail-laminated longitudinal decks previously discussed; however, with stress-laminated decks the load transfer between laminations is developed totally by compression and friction between the laminations, rather than by glue or mechanical fasteners. This friction is created by transverse compression applied to the deck using the same type of high-strength steel-stressing elements that are commonly used for prestressed concrete. These elements, which have historically been high-strength steel rods, are placed at regular intervals through prebored holes in the wide faces of the laminations and are stressed in tension using a hydraulic jack. In a typical stress-laminated lumber deck, each rod may have from 80,000 to 100,000 pounds of tension that is transferred into the deck to develop compression between the laminations. The total force from all prestressing elements on a 32-foot-long bridge, for example, may be as high as 1 million pounds. That 1 million pounds compresses the laminations so tightly that the deck behaves like one large, solid plate of wood (Figure 9-2).

Stress-laminating is the newest development in timber bridge construction and offers many advantages over conventional nail-laminated lumber systems. Deck superstructures can be prefabricated locally into panels, or into complete units, that are shipped to the project site and lifted into place. Once installed, the deck acts as a continuous slab without transverse or longitudinal joints that adversely affect wearing surface performance. In addition, the stress-laminated lumber deck will not delaminate over time, which is a problem associated with nail-laminated lumber construction. Another advantage of stress-laminated decks is the length of lumber required for the laminations. Because load transfer between the laminations is developed from friction, all laminations do not have to be continuous (one piece) over the bridge length. Discontinuous laminations using butt joints are permitted within certain limitations. This provides advan-

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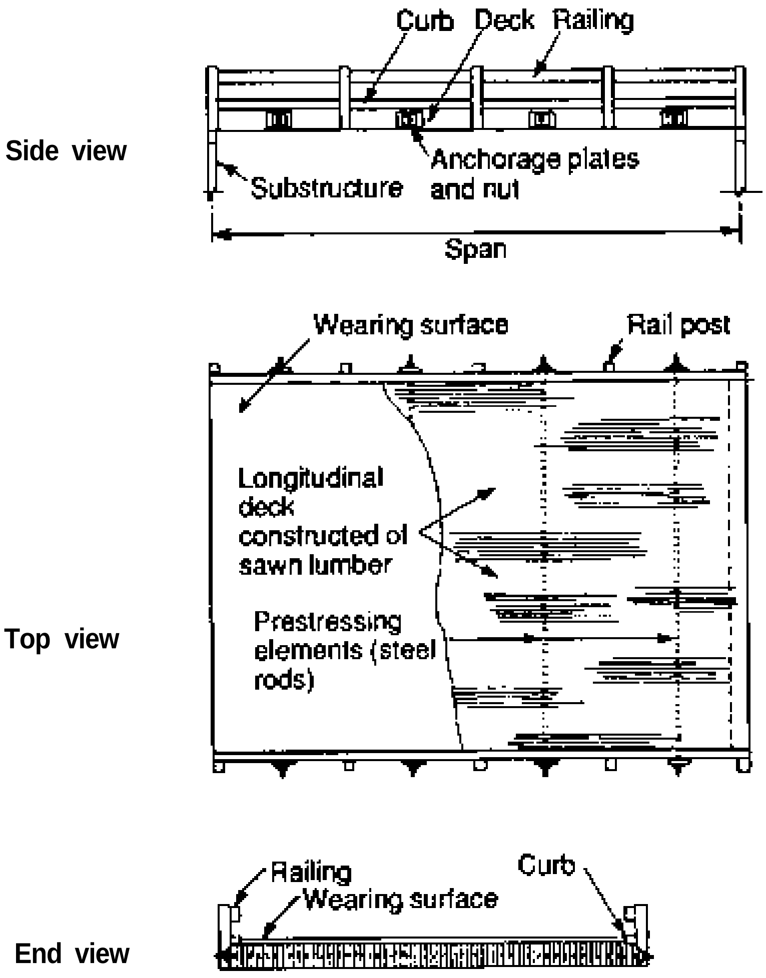


Figure 9-1. - Typical configuration for a longitudinal stress-laminated lumber deck bridge.

Figure 9-2. – Stress-laminated deck bridge built over Iron River on the Cheguamegon National Forest in 1988.

tages over conventional nail-laminated systems because shorter lumber can be used. It also allows longer spans to be cambered to offset dead load deflection.

The concept of stress-laminated lumber was originally developed in Ontario, Canada, in the mid-1970's. Design procedures and specifications were subsequently included in the *Ontario Highway Bridge Design Code* (OHBDC) in 1979.[10,11] Although numerous stress-laminated lumber superstructures have been built in the United States, design provisions are not included in the AASHTO bridge specifications,[1] but are currently being proposed. This chapter presents a brief history of developmental work completed in Ontario and in the United States relative to stress-laminated deck performance and design. The basic characteristics for longitudinal stress-laminated lumber decks are presented and followed by suggested design procedures and examples.

9.2 DEVELOPMENT OF STRESS-LAMINATED BRIDGE SYSTEMS

Stress-laminated lumber has been used as a method of bridge construction for more than a decade. Its inception and development are the result of pioneering efforts in Ontario. Further research and development has occurred in the United States. This section presents a brief summary of the development of stress-laminated lumber bridge systems, including an overview of recent developments in stress-laminating technology and their application to new bridge systems.

DEVELOPMENT IN ONTARIO

Stress-laminating was first used for timber bridges in Ontario in 1976. At that time, the Ontario Ministry of Transportation (MTO, formerly the Ontario Ministry of Transportation and Communication, MTC) was interested in developing new methods for rehabilitating deteriorated nail-laminated lumber bridge decks. Many such decks in Ontario were separating or delaminating under repeated heavy highway loading. Although the static strength and condition of the laminations was good, load distribution between laminations was severely reduced, and the delamination was causing asphalt wearing surfaces to crack and separate from the deck. It appeared to MTO engineers that structural integrity and continuity could be reestablished in the decks by using prestressing techniques to preload and recompress the wood laminae.

In 1976, a pilot project was carried out in Ontario on the Hebert Creek Bridge.[8] This bridge, a longitudinal nail-laminated lumber deck, was in an advanced state of delamination and was scheduled for replacement in 1977. The bridge had an overall length of 55 feet, with the longest span between bents being 20 feet. Steel prestressing rods were placed above and below the existing deck and were tensioned to recompress the deck (Figure 9-3). While stressing the rods, it was found that the compression caused the bridge width to decrease and additional laminations had to be added to maintain the original roadway width. After stressing was complete, MTO load tested the bridge to assess the results.[27] The effects of the stress-laminating were rather dramatic and substantially increased the bridge load-carrying capacity. The rehabilitation method proved so successful that the scheduled bridge replacement was cancelled (more detailed information on the Hebert Creek Bridge and other rehabilitation projects completed in Ontario is presented in Case History 15.5 in Chapter 15).

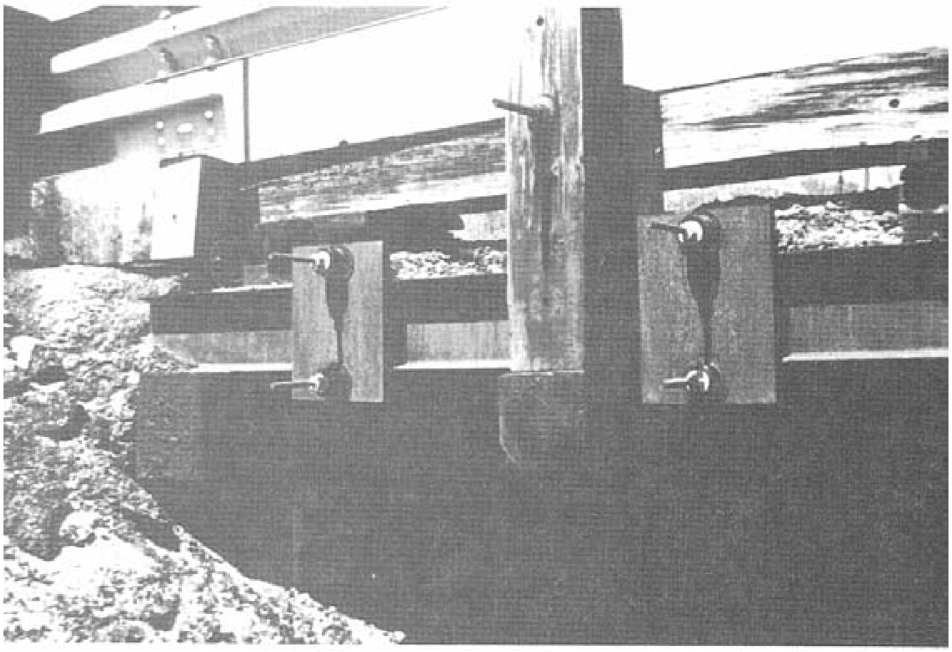


Figure 9-3. - Prestressing rod configuration of the type used on the Hebert Creek Bridge. Rods were placed above and below the existing deck and were anchored to steel plates along the deck edges.

Although the first application of stress laminating in Ontario involved the rehabilitation of an existing bridge, the method offered a variety of possibilities for the construction of new bridges. A long series of development studies was undertaken by MTO and Queen's University to provide an understanding of the fundamentals of stress laminating and to identify possible problems and associated design implications. Among the investigations were tests to determine (1) the friction force developed between the laminations and its dependence on the level of compressive prestress, (2) the mechanism and magnitude of deck bending and deformation, (3) time-related prestress losses, and (4) effective plate stiffness properties of the stress-laminated system. In addition, analytic models were developed to predict deck behavior.

Load testing of small decks in the laboratory of Queen's University proved that stress-laminated lumber decks behave like an orthotropic plate, with different stiffness in the directions parallel to the laminations and perpendicular to the laminations.[6] The stiffness parallel to the laminations was found to depend on the lamination depth and the modulus of elasticity parallel to the wood grain. The transverse system stiffness across the laminations, perpendicular to grain direction, was found to be substantially lower and was expressed as a fraction of the longitudinal stiffness. In comparison to a similar longitudinal glulam deck, the stress-laminated deck showed slightly less transverse stiffness, probably from minor variations in lamination thickness or warp, which reduces interlaminar contact. Thus, a stress-laminated lumber deck is slightly less efficient than a continuous glulam deck of the same size.

Based on research work conducted by MTO and Queen's University, [5,6,7] as well as successful rehabilitation projects in Ontario, a design procedure for stress-laminated decks was developed and included in the 1979 edition of the OHBDC. Subsequently, the system has been successfully used on numerous bridge rehabilitation and new construction projects in Ontario.

DEVELOPMENT IN THE UNITED STATES

Research and development on stress-laminated bridges has been completed in the United States at the University of Wisconsin, Madison (UW) in cooperation with the USDA Forest Service, Forest Products Laboratory (FPL). The focus of this research centered on expanding work done in Ontario and at Queen's University to develop a design procedure for use in the United States. Extensive evaluation and testing were conducted over a 3-year period starting in 1985. During the research, two full-size stress-laminated bridge decks were constructed and tested in the structures laboratory at UW.[9,15,16,17] The first deck was constructed of heavy timber laminations using nominal 4-inch-wide by 16-inch-deep lumber (Figure 9-4). The second deck was built from dimension lumber using nominal 2-inch-wide by 12-inch-deep laminations. Both decks were tested extensively under simulated truck loads for spans up to 48 feet for the heavy timber laminations, and spans up to 24 feet for the dimension-lumber laminations.

The results of the UW/FPL research confirmed many of the Ontario findings and exhibited good correlation with previous truck load tests. The results also were verified for nominal 4-inch-thick laminations, which had not been previously tested. In addition, UW/FPL research investigated new areas of stress-laminated deck behavior, including (1) the effects of lamination butt joints on wheel-load distribution and deck stiffness, (2) the mechanism of stress transfer into the deck and related edge effects on wheel-load distribution, (3) the effects of transverse bending on the required level of compressive prestress, and (4) requirements for anchorage of prestressing rods (which resulted in a new anchorage design without the

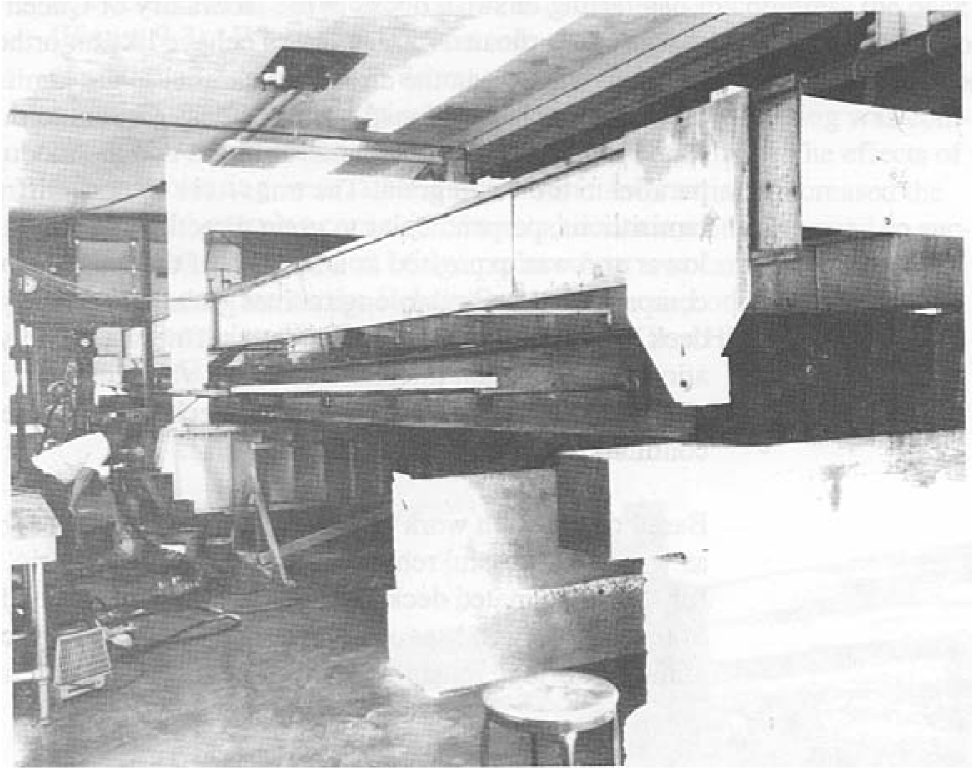


Figure 9-4. - Full-scale experimental stress-laminated deck in the structures laboratory at the University of Wisconsin.

steel channel bulkhead traditionally used in Ontario). Additional research is continuing at FPL and at other universities in the United States to substantiate the performance of prototype stress-laminated deck bridges. Cooperative work between West Virginia University and FPL is currently in progress to develop design procedures and performance characteristics for stress-laminated decks constructed from native hardwood species. A long-term moisture study also is being conducted by FPL to determine the effects of moisture variations in the laminations on the level of compressive prestress.

To date, nearly 20 stress-laminated decks have been built in the United States. Many of these bridges are being periodically monitored and load-tested to assess field performance and verify design criteria (Figure 9-5). In 1989-1990, approximately 60 new stress-laminated bridges will be constructed through the USDA Forest Service Timber Bridge Initiative; Approximately half of these bridges will be built in West Virginia under the supervision of West Virginia University. The remainder will be distributed across more than 20 states. Data obtained from monitoring these bridges will provide a great deal of information on stress-laminated deck performance in a wide range of environmental conditions.

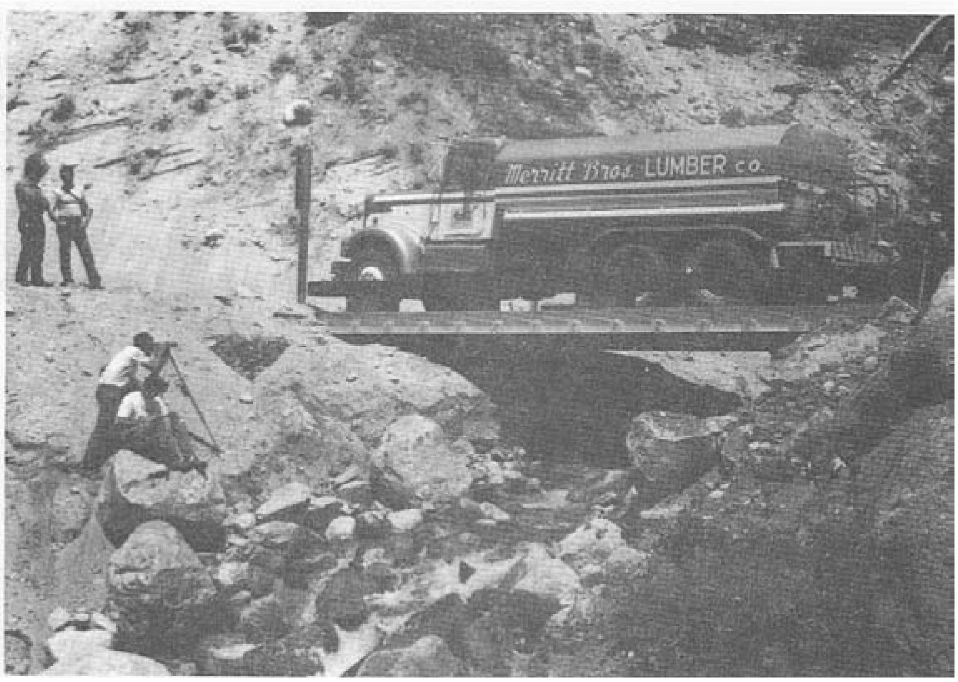
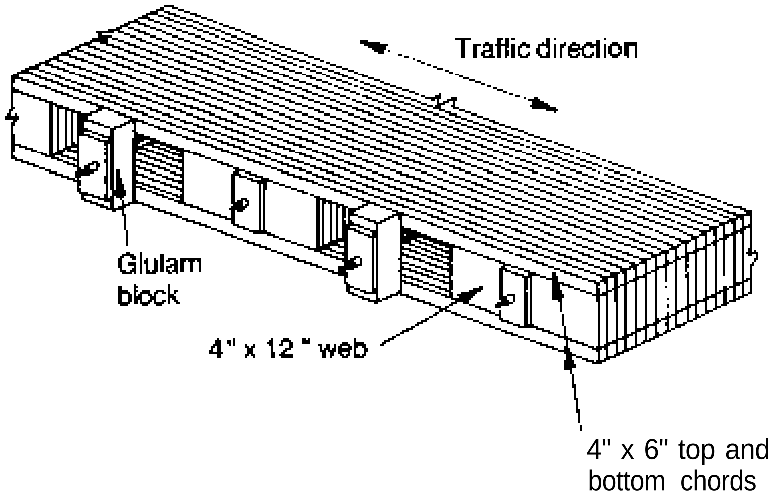


Figure 9-5. - Load test of the Zuni Creek stress-laminated deck on the Idaho Panhandle National Forests.

NEW STRESS-LAMINATED SYSTEMS

The stress-laminated bridge investigations previously described have involved the use of longitudinal sawn lumber laminations with transverse prestressing (some projects in Ontario have used transverse lumber laminations with longitudinal prestressing). Although these designs have proved successful in short-span applications, the moment of inertia of the deck is limited by the available depth of lumber laminations, which is generally 16 inches nominal. Like other longitudinal deck systems constructed of glulam or nail-laminated lumber, span capabilities of longitudinal stress-laminated decks are normally controlled by stiffness (deflection), rather than stress. The need for longer spans has focused attention on developing new designs for stress-laminated timber bridges that provide additional stiffness. Although these new systems are in a developmental stage at this time, and no design criteria or procedures are available, design criteria should be forthcoming.

Work has recently been completed at UW/FPL on a new bridge system using parallel-chord trusses that are stress-laminated together.[9,19] By using parallel-chord trusses in place of the sawn lumber laminations, a deeper, stiffer system was obtained using the same volume of lumber. A full-size, 52-foot span, stress-laminated parallel-chord bridge was built and tested in the UW structures laboratory under various simulated loading conditions. Individual truss laminations consisted of 4-inch-wide by 6-inch-deep top and bottom chords, separated by 4-inch-wide by 12-inch-deep discontinuous web members (Figure 9-6). The connection between the top and bottom chord and the web was made with steel-drive spikes that were placed through the chords, into the web. The stress-laminated trusses produced a significant increase in bridge stiffness compared to sawn lumber laminations, yet exhibited many of the same characteristics previously observed for longitudinal decks. After laboratory testing, a prototype stress-laminated parallel-chord truss bridge was built by the Forest Service on the Hiawatha National Forest in late 1987. This structure has been load tested on two occasions and is being continuously monitored.



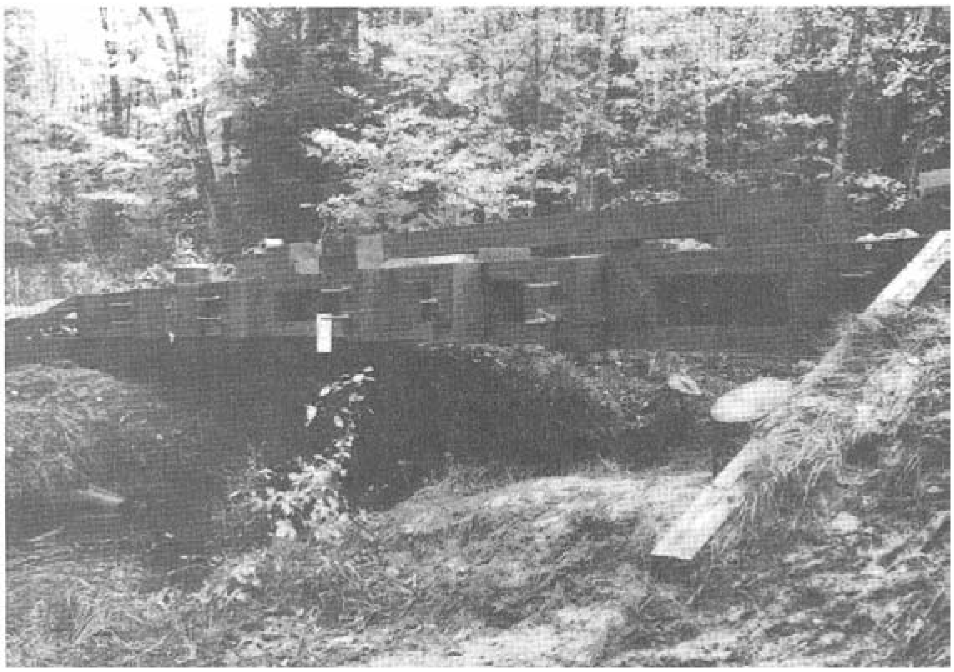


Figure 9-6. - (Top) Drawing of a stress-laminatedparallel-chord truss. (Bottom) Prototype stress-laminated parallel-chord bridge built on the Hiawatha National Forest (shown during construction).

In addition to the parallel-chord truss work done by UW/FPL, new applications of stress-laminating are being investigated in Ontario and at several universities in the United States. Ontario is investigating the development of a stress-laminated cellular or box girder-type of bridge, the advantages of which have already been recognized in steel and concrete bridge construction. If the Ontario development work is successful, the cellular stress-laminated wood system may be very competitive with other systems for longer-span applications. West Virginia University also has performed laboratory tests and has constructed a prototype bridge using a T or ribbed cross section. In this design, deep, laminated veneer lumber (LVL) beams are stress-laminated to a relatively thin, sawn lumber deck (Figure 9-7). The ribs formed by the deeper LVL laminations contribute substantially to the longitudinal bridge stiffness, making longer spans possible. A similar system using glulam rather than LVL beams also is feasible and is being developed. Other cooperative work between West Virginia University and FPL is aimed at developing stress-laminated box girder systems and new methods for adapting stress-laminated decks to other bridge superstructures constructed of glulam, steel, or concrete. In addition, Pennsylvania State University is developing a stress-laminated wood-steel composite bridge system that is intended to increase bridge stiffness and reduce long-term deflection.

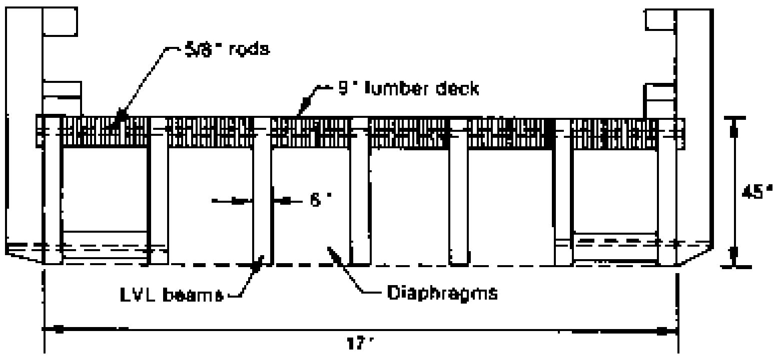
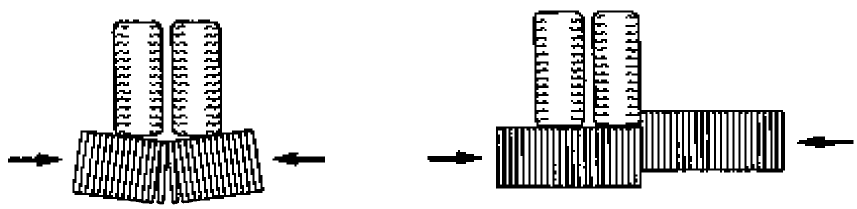


Figure 9-7. - Drawing of a stress-laminated T-bridge cross section.

9.3 CHARACTERISTICS OF LONGITUDINAL STRESS-LAMINATED LUMBER DECKS

As previously discussed, stress-laminating creates a large plate of wood that is held together by compressive forces applied through the prestressing elements. When subjected to vehicle loading, a stress-laminated bridge deck acts as an orthotropic plate with different properties in the longitudinal and transverse directions. When a wheel load is placed at any point on the deck, the entire deck deflects downward (except at locations over the supports), resulting in displacements in both the longitudinal and transverse directions. Because of this behavior, bending moments are also developed in the longitudinal and transverse directions. The magnitude of these moments depends primarily on five variables: (1) load magnitude, (2) deck span, (3) deck width, (4) longitudinal deck stiffness, and (5) transverse deck stiffness. The longitudinal bending moment produces bending stress and deflection that controls the required deck thickness. The transverse moment, which also produces bending stress and deflection, dictates the amount of compressive prestress that must be applied between the laminations.

When a truck wheel load is placed over the deck laminations, two primary actions occur that deteriorate the platelike behavior of the deck (Figure 9-8). The first action results from transverse bending, which produces a tendency for opening between the laminations on the deck underside. The second type of action is from transverse shear, which creates a tendency for the laminations to slip vertically. In both cases, the actions will not occur if the deck has a sufficient level of compressive prestress between the laminations. In the case of transverse bending, the compressive stress directly offsets the tension effects on the deck underside. For shear, vertical slip is prevented by friction between the laminations resulting from the compressive prestress. Maintaining an adequate level of prestress is the single most important aspect of stress-laminated bridge construction.



Transverse bending produces a tendency for opening between the laminations on the deck underside.

Transverse shear produces a tendency for laminations to slip vertically.

Figure 9-8. - Actions that tend to reduce the platelike behavior of a longitudinal stress-laminated bridge deck.

Stress-laminating is a relatively new concept for bridge construction in the United States. Although there have been numerous stress-laminated bridges built in this country, information about system characteristics and design requirements are not as widely available as they are for other, more conventional timber bridge systems. Many aspects of longitudinal stress laminated deck design are similar to those for other longitudinal deck systems, but several characteristics are unique to stress laminating. The most important of these characteristics are related to the lumber laminations, prestress elements and anchorages, time-related stress loss, and construction methodology.

LUMBER LAMINATIONS

The lumber laminations of a stress-laminated bridge provide the required strength and stiffness for bridge performance and serviceability. Of particular interest are characteristics related to material requirements, load sharing, and lamination joints.

Material Requirements

Longitudinal stress-laminated bridges are constructed from visually graded or MSR lumber in the Joists and Planks size classification (nominally 2 to 4 inches thick, 5 inches and wider). Although decks could theoretically be constructed from any lumber thickness, the 2- to 4-inch thickness range has proven most efficient and economical. The laminations may be dressed, rough-sawn or full-sawn; however, rough-sawn and full-sawn material must be surfaced to a uniform thickness to ensure even bearing between the laminations. To date, most bridges constructed in the United States have used rough-sawn 4-inch nominal lumber that is surfaced on one side (S1S) to provide a uniform thickness.

Stress-laminated bridge decks can generally be built from any lumber species provided it meets design requirements for strength and stiffness and is treatable with preservatives. At this time, however, the number of suitable species is somewhat limited because parameters for stress-laminated deck design have not been established for all species. Research has been completed for Douglas Fir-Larch, Hem-Fir (North), Red Pine and Eastern White Pine.[5,6,9] Research for other species is currently in progress and will be available in the near future. For all species, the lumber laminations used for stress-laminated decks should be treated with oil-type preservatives (Chapter 4). As previously discussed for other bridge types, the oil-type preservatives provide a protective barrier that helps reduce wood moisture content variations and associated dimensional changes. This is especially important for stress-laminated construction because dimensional changes in the laminations can affect the level of compressive prestress in the bridge.

Load Sharing

When lumber laminations are stressed together, the strength-reducing characteristics of the individual laminations are dispersed throughout the cross section in the same manner previously discussed for glulam (Chapter 3). Like glulam, the bending strength of stress-laminated lumber is substantially greater than a comparable sawn lumber member of the same size. Research conducted in Canada showed that stress laminating increases usable bending strength by 50.8 to 82.5 percent, depending on the grade and species of lamination.[26,29] For stress-laminated bridges, the

OHBDC currently allows a bending stress increase of 50 percent for lumber of mixed grades No. 1 and No. 2, and 30 percent for lumber graded Select Structural.

Lamination Joints

As previously discussed, load transfer between laminations in a stress-laminated bridge is accomplished by friction between the laminations induced by the high level of compressive prestress. Because this friction is sufficient to prevent movement between the laminations, it can be used as a means of longitudinally splicing the laminations. Thus, the laminations for a stress-laminated bridge deck need not be continuous over the bridge span and can be provided with longitudinal butt joints. When butt joints are used, the OHBDC requires that not more than one butt joint occur in any four adjacent laminations within a 4-foot distance, measured along the bridge span (Figure 9-9).

The ability to use butt joints in stress-laminated decks provides an advantage over conventional nail-laminated construction because shorter laminations can be used, resulting in reduced costs and improved availability. However, research at UW has shown that butt joints reduce longitudinal stiffness, and therefore must be compensated for in design. In addition, the discontinuity at the joint reduces the effective deck section available to resist bending stress. The effects of butt joints are discussed further in the design procedures given later in this chapter.

PRESTRESSING SYSTEMS

The prestressing system is perhaps the most important part of a stress-laminated bridge because it holds the bridge together and develops the necessary friction between the laminations. The system generally consists of two parts: the prestressing elements and the anchorages. The prestressing elements are placed transverse to the bridge span and are stressed in tension. The anchorages hold the prestressing elements along the deck edges where the tension is transferred into the lumber lamina-

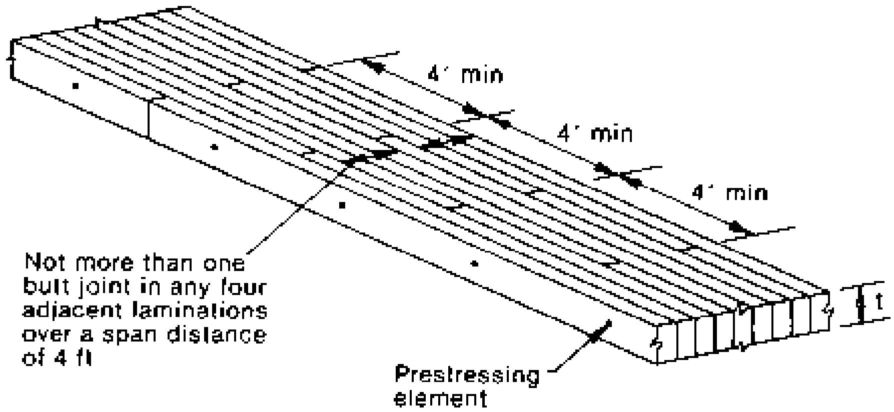


Figure 9-9. - Minimum requirements for butt joints in longitudinal stress-laminated bridge decks.

tions. The function of the prestressing system is to develop the required uniform compressive force between the laminations. Research at UW has shown that the compressive prestress is localized at the anchorage, but becomes uniformly distributed at interior locations, away from the anchorage.

Prestressing Elements

Prestressing elements for stress-laminated bridge decks must be carefully selected for their strength and corrosion-resistance properties. All stress-laminated bridges constructed to date have used high-strength threaded steel rods that conform to ASTM A 722, Uncoated High-Strength Steel Bar for Prestressing Concrete.[4] These rods have a minimum ultimate stress in axial tension of 150,000 lb/in^2 and are available in diameters ranging from 5/8 inch to 1-3/8 inch. Steel prestressing strand has not been used but is being investigated and may prove to be an alternate material in the future. The potential advantages of strand include its higher strength (270,000 lb/in^2 ultimate tensile stress) and lower cost. A disadvantage with strand is that the anchor chuck damages the strand so that it cannot be restressed. As a result, the strand must be replaced each time the bridge is restressed.

Because steel prestressing elements are under high stress, and are particularly susceptible to corrosion, it is essential that special corrosion protection be provided. Existing applications have predominantly used galvanizing to protect the rods and this method should be used until alternative techniques are proven. Galvanizing is generally provided by the rod manufacturer using processes that avoid embrittlement or strength loss in the high-strength steel. Epoxy coatings, similar to those used for concrete-reinforcing steel, are being evaluated and have been used with good results in several applications. In addition, some bridge rehabilitation applications in Ontario have successfully used plastic (PVC) pipes that are placed over the rods and are filled with grease (see Case History 15.5 in Chapter 15).

Anchorages

The anchorages for prestressing elements must transfer the required stress to the lumber laminations without causing wood crushing in the outside laminations. Additionally, they must be capable of developing the full capacity of prestressing elements. Anchorage systems for steel prestressing rods have traditionally used steel plates or shapes. The rod is placed through the steel components and anchored with a nut. The nuts match the coarse thread pattern on the rods and are made from high-strength steel by the rod manufacturer. Standard nuts are not compatible with high-strength stressing rods.

Two types of anchorages are used for longitudinal stress-laminated decks; one for deck rehabilitation where rods are placed externally, over and under the lumber laminations, and one for new deck construction where rods are placed internally, through holes in the laminations. For bridge rehabilitation, the external channel bulkhead anchorage was developed in Ontario and employs a continuous steel channel along the deck edges (Figure 9-10 A). The rods extend beyond the channel and are attached with nuts to rectangular steel anchorage plates. For new construction, two anchorage configurations are currently used: the channel bulkhead configuration and the bearing plate configuration. The channel bulkhead configuration was developed in Ontario and is currently a design requirement in the OHBDC (Figure 9-10 B). It is similar to the external channel bulkhead used for deck rehabilitation, but the rods extend through the center of the channel and attach to rectangular steel bearing plates along the channel web. Although considered necessary for bridge rehabilitation, research at UW showed that the steel channel contributed little to load transfer or bridge performance for new construction. A new anchorage employing a large, rectangular steel bearing plate and a smaller, outside anchorage plate was developed by UW/FPL (Figure 9-10 C). Most of the stress-laminated deck bridges constructed in the United States have substituted this steel-plate configuration for the continuous channel. It should be noted, however, that although the steel channel is not considered necessary from a structural standpoint, in certain circumstances it may be desirable to cover the outside laminations. In past applications, the steel plates used without channels have caused some local wood crushing and created an indentation in the outside laminations. The channel effectively covers these areas so they are not visible.

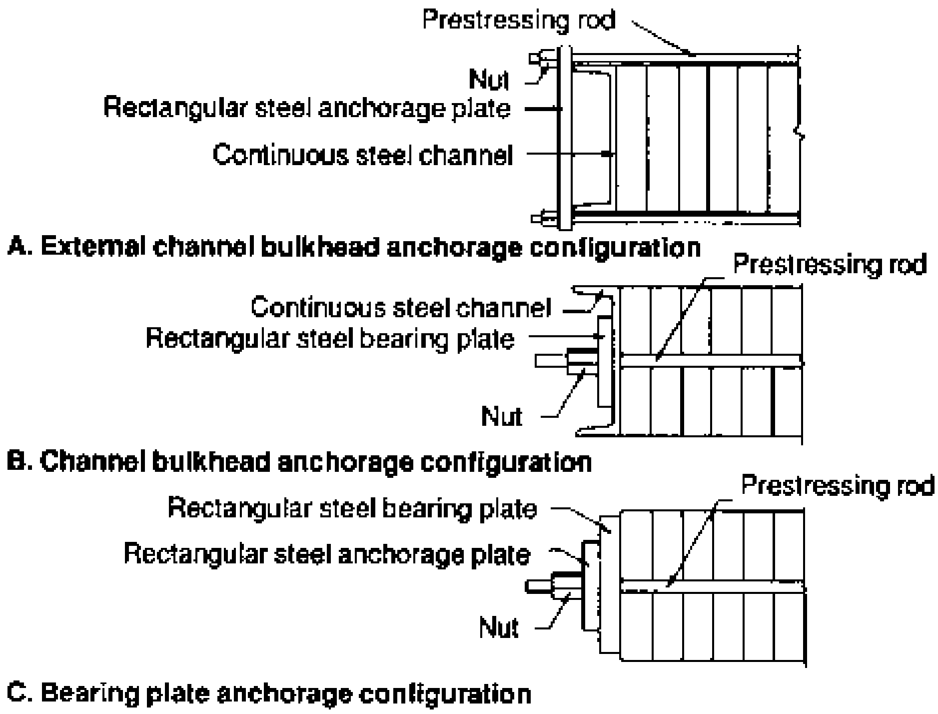


Figure 9-10. - Types of anchorages for steel prestressing rods.

TIME-RELATED STRESS LOSS

A sufficient level of uniform, compressive prestress must be maintained between the lumber laminations in order for a stress-laminated bridge to perform properly. With time, the initial level of prestress placed in the deck at installation will be affected primarily by two factors: creep in the wood and variations in wood moisture content. The Ontario research proved that when a constant compressive force is applied to wood over time, the wood slowly compresses or creeps. This occurs because the wood cells gradually change shape and become permanently compressed. Thus, when the deck laminations are compressed by the prestressing force, they slowly become narrower. Unfortunately, the level of prestress decreases when this occurs. Work in Ontario found that this loss of compression from creep increased when the cross-sectional area of the steel prestressing components increased. To reduce this loss effect, it was found necessary to use high-strength steel rods to carry the large prestressing force with a minimum cross-sectional area of steel. In addition, design limits were placed on the ratio of the wood area to the steel area (discussed in the next section on design).

Although creep is a natural wood characteristic that adversely affects the compressive prestress level, the research done in Ontario has developed a method of effectively controlling this phenomenon. Specifically, the amount of creep in a stress-laminated deck was found to be directly related to the number of times the deck is stressed (Figure 9-11). If a deck is stressed only once during construction, 80 percent or more of the initial compression may be lost to creep. If the deck is restressed within a relatively short period, the subsequent stress loss is less. If the deck is restressed a second time within a specified time period, the total compression loss over time can be limited to a maximum of 60 percent. Research at UW/FPL found that a stress-laminated deck would perform acceptably at a compressive prestress level as low as 24 lb/in^2. Because this is many

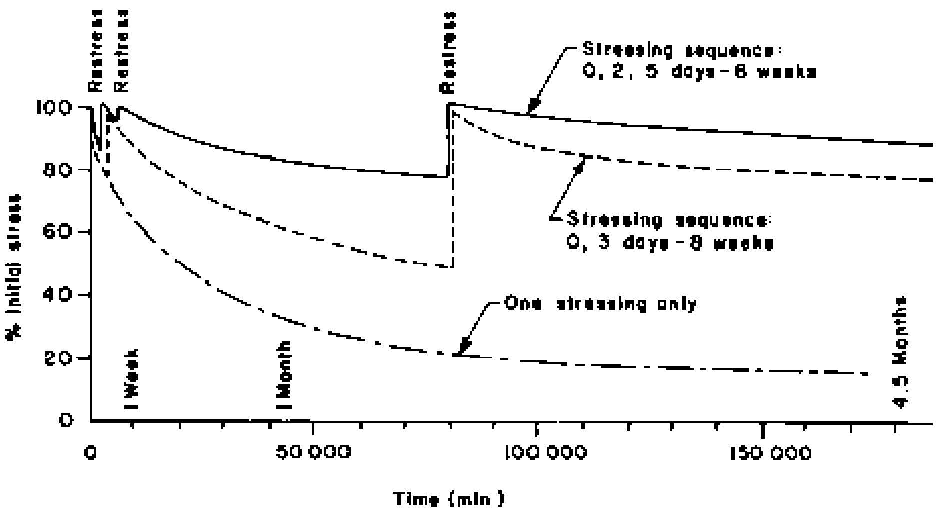


Figure 9-11. - Effects of restressing on time-related stress loss (from Csagoly and Taylor).[8]

times lower than the strength of the wood in compression perpendicular to grain, the level of compressive prestress placed in a bridge during the initial stressing operations is increased to compensate for subsequent creep losses over the life of the structure (the actual prestress level depends on a number of factors discussed later in the design procedures). Thus, a subsequent stress loss from creep of 60 percent over the life of the bridge will still leave the minimum prestress level required for acceptable performance, plus an additional margin for safety. To maintain the minimum prestress level, the following stressing sequence is used:

1. The deck is initially assembled and stressed to the design level required for the structure.

2. The deck is restressed to the full level approximately 1 week after the initial stressing.

3. Final stressing is completed 4 to 6 weeks after the second stressing.

When this stressing sequence is followed, the maximum expected loss in prestress from creep will be limited to approximately 60 percent of the initial level (40 percent of the initial stress level will be maintained). It may be desirable, however, to periodically recheck stress levels over the life of the structure as part of a preventative maintenance program.

In addition to stress loss from creep, the prestress level in stress-laminated lumber decks can be affected by variations in the moisture content of the lumber laminations. As discussed in Chapter 3, wood below the fiber saturation point (approximately 30 percent moisture content) shrinks when moisture is lost and expands when moisture is gained. The effects of these moisture changes can result in a loss or gain in prestress. Research on moisture-related stress changes has involved a few laboratory tests and periodic monitoring of bridges installed in different environmental conditions. Although some changes in prestress have been observed, they have been relatively minor when the lumber laminations were dry (less than 19 percent moisture content) at the time of construction. When lumber is not dry at the time of construction, some bridges have shown an increased loss in prestress as the lumber dries in service. At this time, moisture effects have not been determined to be an important consideration for stress-laminated bridges when dry lumber is used. When lumber with a moisture content above 19 percent is used, periodic restressing may be required until the lumber laminations reach equilibrium moisture content. An evaluation of moisture effects for various lumber species and preservative treatments is under way which will provide insight into the potential for stress changes related to moisture.

CONSTRUCTION METHODOLOGY

Several characteristics related to the construction of stress-laminated decks are unique compared with other bridge systems. Although many of the general principles of timber bridge construction apply to stress-laminated construction, unique methodology is involved in the areas of bridge assembly, camber, and stressing. A brief description of these topics is presented below. A more complete description of the construction of a stress-laminated lumber deck is presented in case histories given Chapter 15.

Bridge Assembly

Stress-laminated bridges can be assembled using three different methods. Two methods involve on-site assembly, while the third involves preassembly at a fabrication facility. The first on-site method involves assembly over the abutments or intermediate supports. Using this method, the laminations are sequentially placed and aligned, and the prestressing rods are inserted and stressed. This method is generally acceptable when the laminations span the full distance between supports and there are no butt joints. When laminations with butt joints are used, scaffolding or other temporary supports must be used to support the laminations until the bridge is stressed. As a result, this method is seldom practical when butt joints are used.

Another option for on-site assembly is to assemble the bridge at a staging area adjacent to the crossing, then lift the entire deck into place. This method offers some advantage over the previous method because the laminations can be supported by the ground rather than by scaffolding. A disadvantage, however, is that a crane or other equipment is required to lift the bridge into place. For both on-site assembly methods, all stressing must be accomplished in the field. After initial construction, two additional trips must be made to the site to complete the required stressing sequence.

In many applications, the preferable method of bridge assembly involves prefabrication at a manufacturing or fabrication facility. With this method, the bridge is fabricated in a series of stressed panels that are normally 7 to 10 feet wide, depending on transportation restrictions and lifting capacity at the site. The panels are shipped to the bridge site, lifted into place, and stressed together to form a continuous deck. To join the panels, the stress in alternate opposing rods is released and the anchorage plates on the joint edge of the released rods are removed (Figure 9-12). The released rods are then inserted into a special coupler on the opposite stressed rod, and the two panels are stressed together (see Case History 15.9 in Chapter 15). Most stress-laminated lumber bridges constructed in the United States have utilized prefabricated panels. The method has been most economical and requires a minimum time for field erection. Another advantage of using the prefabricated panel method is that the restressing sequence can be completed at the fabrication facility. Repeated trips to the bridge site for restressing are not required.

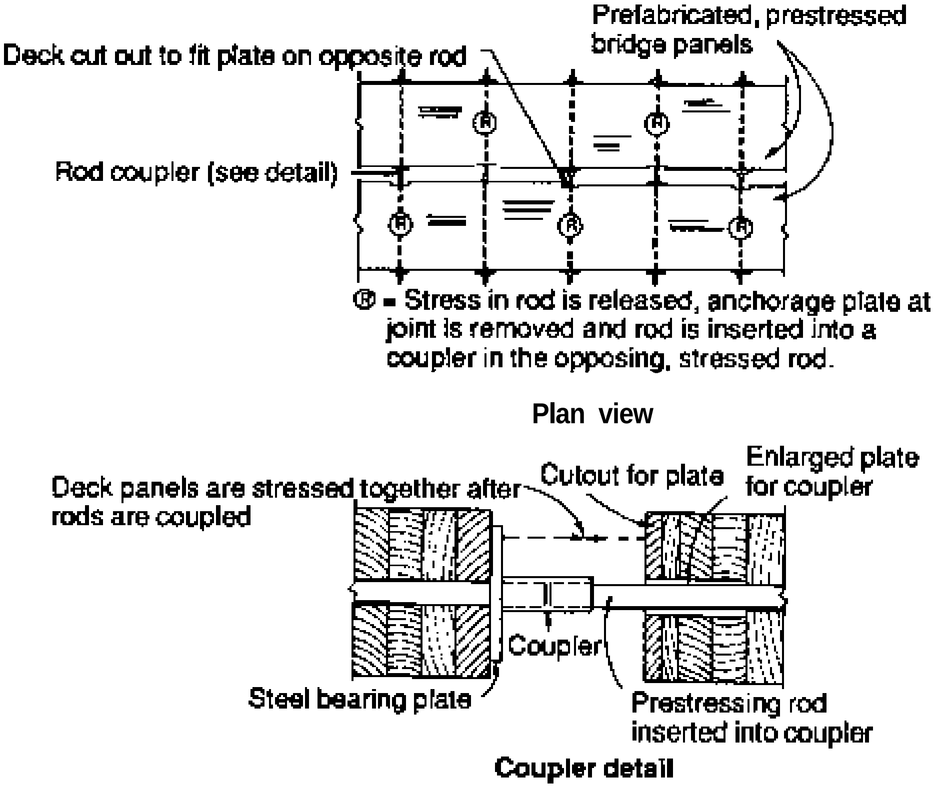


Figure 9-12. - Method of joining two prefabricated, prestressed longitudinal stress-laminated bridge panels.

Camber

Camber is an upward curvature that is placed in a bridge to offset vertical dead load deflection. Stress-laminated decks are unique among timber decks because when butt joints are used, the deck can be cambered (lumber decks without butt joints cannot be cambered). Cambering is accomplished by slightly offsetting the laminations at butt joints before stressing. When the deck is prefabricated or assembled on the ground, this is done with sleeper blocks that are placed under the laminations (Figure 9-13). If the bridge is assembled on scaffolding, the same effect is achieved by

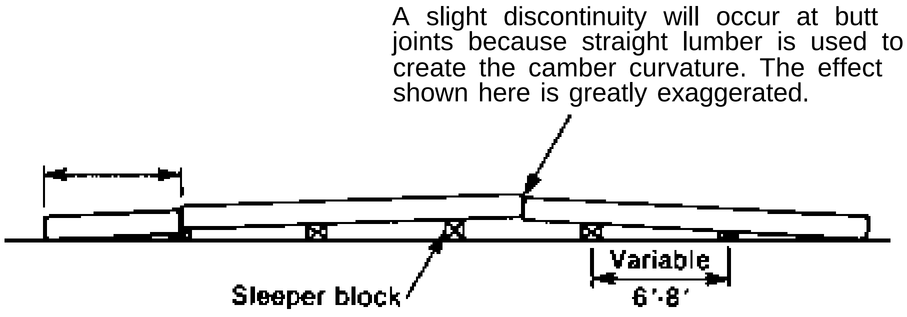


Figure 9-13. - Method of cambering longitudinal stress-laminated bridge joints with butt joints.

varying scaffolding height. After the desired amount of curvature is built into the deck, it is stressed together and the camber is locked in. Because stress-laminated decks use straight lumber, cambering causes slight discontinuities at the butt joints. However, these discontinuities are normally of little consequence. The amount of centerspan camber recommended for stress-laminated decks is a minimum of two times, and preferably three times, the deck dead load deflection.

Stressing

Stress-laminated lumber decks are stressed together with a hydraulic jack that applies tension to the prestressing rod by pulling the rod away from steel anchorage plates. After the tension is applied, the nut is tightened against the anchorage plate and the tension remains in the rod when jack pressure is released. Two types of jacks have been used for stress-laminated decks, both of which are hollow-core jacks (the prestressing rod is inserted through the jack body)(Figure 9-14). The first type uses a built-in ratchet to tighten the nut after stress has been applied. The second type involves a standard hollow-core jack used with a prefabricated steel chair. The rachet-type jacks are available from rod manufacturers and are simple and convenient to operate; however, they are expensive to purchase or rent. The hollow core and steel chair arrangement is much less expensive, but the nut must be tightened with a wrench rather than a built-in ratchet.

The method used for stressing a stress-laminated lumber deck depends on the number of jacks that are available. In Ontario, bridges have generally been stressed using a series of up to 24 jacks. Although it is expensive to purchase or rent a large number of jacks, this method is most convenient because the entire deck can be stressed in one operation. In the United States, most stress-laminated lumber decks have used a single jack that is sequentially used for each rod. When using the single-jack method, jacking starts at the first rod on one end of the bridge and progresses to the last rod on the opposite end. After all rods are stressed the first time, three or more additional passes are necessary to restress each rod to the required level. This restressing is necessary because the initial stress in one rod squeezes the laminations together and reduces the stress in adjacent rods. In most cases, the proper uniform stress is achieved by making four passes along the deck.

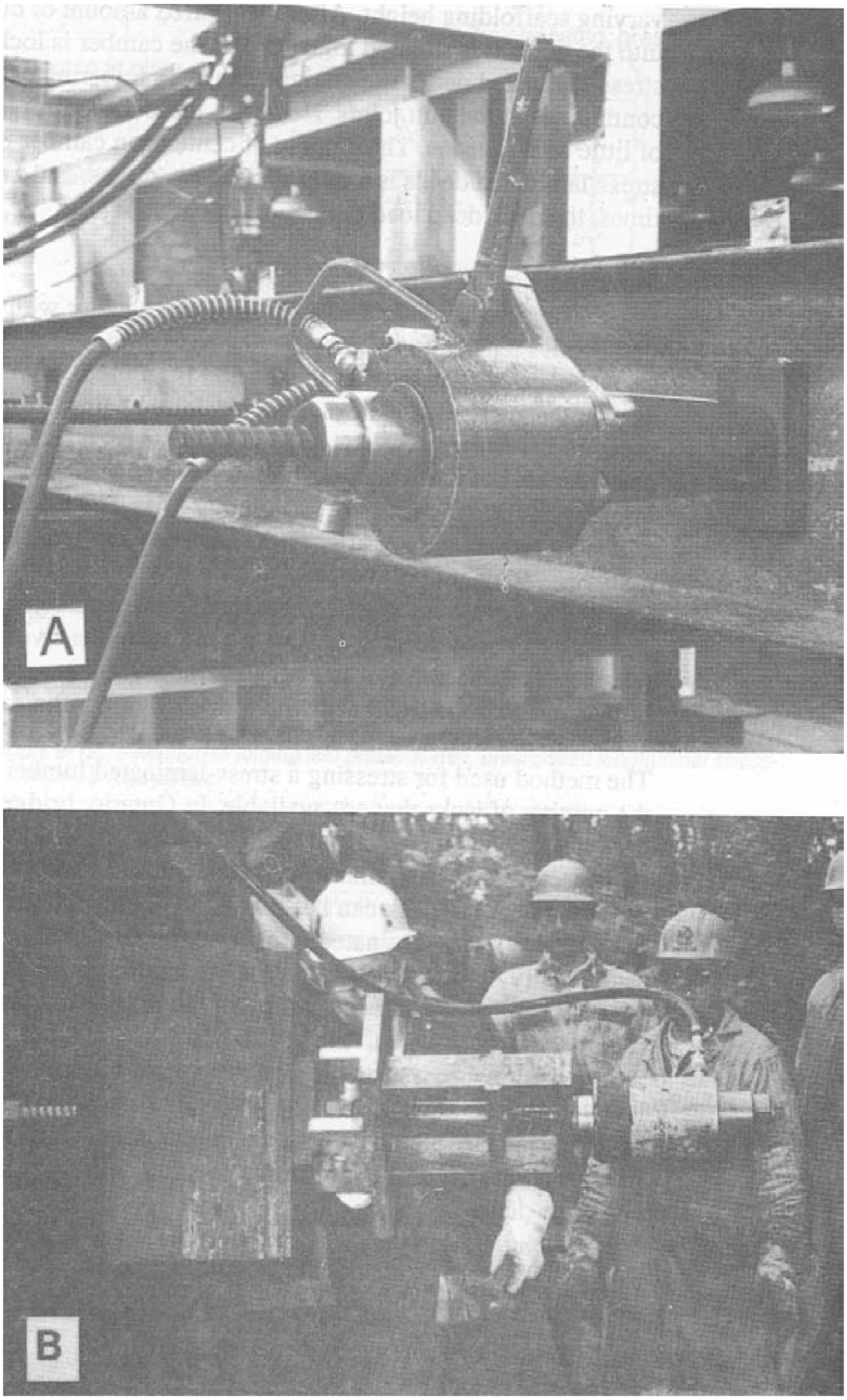


Figure 9-14. - Types of hollow core jacks used for stress-laminated bridges. (A) With a built-in ratchet. (B) With a steel chair assembly.

9.4 DESIGN OF LONGITUDINAL STRESS-LAMINATED LUMBER DECKS

The design of longitudinal stress-laminated lumber decks is controlled by four basic design constraints. The first and most obvious constraint is ensuring safety by limiting the material to allowable stresses that provide an acceptable factor of safety. The second constraint involves maintaining sufficient stiffness within the deck to avoid long term sagging and unacceptable live load deflection. The third constraint requires that the necessary minimum uniform level of compressive prestress be maintained to keep the bridge laminated together over the design life. Finally, the stress induced by the prestress compression must be within acceptable limits to avoid wood damage.

This section presents sequential design procedures and examples for longitudinal stress-laminated lumber decks. As previously discussed, design provisions for stress-laminated lumber are not included in current AASHTO bridge design specifications, although they are currently being developed. The design procedures presented here are from a preliminary AASHTO proposal based on laboratory and field research conducted by UW/FPL. In addition, provisions from the *Ontario Highway Bridge Design Code* are included, based on research completed by Queen's University and MTO. Other design procedures currently being developed at West Virginia University and at MTO will be considered by the appropriate AASHTO committees when research is completed. The basis for the procedures proposed by West Virginia University, including equations for deflection and bending moment, are presented in the West Virginia University Civil Engineering Report *Wheel Load Distributions on Highway Bridges.*[20]

DESIGN CRITERIA AND DEFINITIONS

General design requirements related to stress-laminated deck design are summarized below. Additional criteria related to specific component design are addressed in more detail in the design procedures and examples that follow.

Deck Configuration

The following limitations apply to stress-laminated lumber decks designed using this procedure:

1. The deck is constructed of sawn lumber laminations that are placed edgewise between supports and are transversely stressed together.

2. Deck width is constant.

3. Deck thickness is constant and is not less than 8 inches nominal.

4. The deck is a rectangle in plan, or is skewed less than 20 degrees.

5. End or intermediate supports are continuous across the deck width.

6. Butt joints are permitted in the laminations provided no more than one butt joint occurs in any four adjacent laminations within a span distance of 4 feet.

Loads

Loads are based on AASHTO loading requirements discussed in Chapter 6. Design procedures and examples are limited to AASHTO Load Group I and IB, where design is routinely controlled by a combination of structure dead load and vehicle live load. As with other timber bridge types, allowable design stresses may be increased by 33 percent for overloads.[2] AASHTO special provisions for H 20-44 and HS 20-44 wheel loads (Chapter 6) do not apply to longitudinal stress-laminated decks.

Lumber Laminations

Design procedures are valid for sawn lumber laminations of Douglas Fir-Larch, Hem-Fir (North), Red Pine, or Eastern White Pine. Behavior and performance data on other species are not currently available but are being developed. Conditions of use are based on a normal duration of load and wet-use conditions, without adjustments for temperature and fire-retardant treatments. *All wood components are assumed to be pressure treated with an oil-type preservative prior to fabrication.*

Tabulated values for lumber are taken from the 1986 edition of the NDS.[12] To account for load-sharing characteristics of the stress-laminated system, the tabulated bending stress for single-member use is increased 30 percent for lumber graded Select Structural, and 50 percent for lumber graded No. 1 or No. 2. These increases are based on research conducted in Canada (discussed in Section 9.3) and are somewhat less than load-sharing increases currently allowed in the United States for glulam.

Prestressing System

Prestressing elements are high-strength steel rods conforming to ASTM A722.[4] The rods are placed through the laminations and are attached to anchorages with high-strength nuts (refer to OHBDC for design requirements related to rod configurations placed above and below, rather than through, the laminations). Design procedures are included for both the steel plate anchorage and the channel bulkhead anchorage. Either system may be used at the prerogative of the designer. All prestressing components and metal hardware are galvanized or otherwise provided with acceptable corrosion protection.

Live Load Deflection

AASHTO specifications do not include design criteria or guidelines for live load deflection in timber bridges. The recommendations given in this section are based on field experience and common design practice, and are consistent with recommendations previously given for other timber bridge types. Although it is highly recommended that these maximum-deflection guidelines be followed for best performance, deflection criteria should be based on specific design circumstances and are left to designer judgment.

DESIGN PROCEDURES

The design of longitudinal stress-laminated decks is basically a two-part process involving design of the lumber laminations followed by design of the prestressing system. Lamination design is based on a wheel load distribution width similar to that used for longitudinal nail-laminated decks. Using this approach, the deck is assumed to act as a beam and is designed for bending, deflection, and compression at the supports. Horizontal shear is not a controlling factor in stress-laminated deck design, and need not be considered. Design of the prestressing system is based on the deck configuration and the magnitude of the transverse moment and shear. For both the deck and the prestressing system, design procedures use graphs that are based on variable relationships developed by analytic modeling, verified by full-scale structure performance.

The basic design procedures for longitudinal stress-laminated lumber decks are outlined in the following steps. The sequence of the procedures assumes that the deck thickness is initially based on bending, then checked for deflection. In many applications, deflection will control; however, the acceptable level of deflection may vary for different design applications. The order of the procedures may be rearranged as necessary.

1. Define deck geometric requirements and design loads.

a. Define geometric requirements for bridge span, width, and the number of design traffic lanes. The effective deck span, *L*, is the distance measured center to center of supports. Deck width is the roadway width plus additional width required for curb and railing systems.

b. Identify design vehicles (including overloads), other applicable loads, and AASHTO load combinations discussed in Chapter 6. Also note design requirements for live load deflection and other site-specific requirements for geometry or loading.

2. Select a species and grade of lamination and compute allowable design values.

Stress-laminated decks are normally constructed from lumber in the Joists and Planks size classification (2 to 4 inches thick, 5 inches and wider). Grades for visually graded lumber are generally No. 2 or better for nominal 2-inch material and No. 1 or better for nominal 4-inch material. Select a species and grade of lumber from the NDS Table 4A (Douglas Fir-Larch, Hem-Fir (North), Red Pine, or Eastern White Pine) and compute allowable design values for bending (F_b'), modulus of elasticity (E'), and compression perpendicular to grain ($F_{c\perp}'$) by Equations 9-1, 9-2, and 9-3, respectively. Tabulated single-member bending stress given in the NDS Table 4A is increased by the load-sharing factor, C_{LS}.

$$F_b' = F_b C_M C_{LS} \tag{9-1}$$

$$E' = EC_M \tag{9-2}$$

$$F_{c\perp}' = F_{c\perp} C_M \tag{9-3}$$

where C_M = moisture content factor from Table 5-7, and

C_{LS} = load sharing factor (1.30 for lumber graded Select Structural, 1.50 for lumber graded No. 1 or No. 2).

3. Determine preliminary lamination layout.

The design of stress-laminated lumber decks depends on the configuration of the laminations and the frequency of butt joints. Butt joints reduce the required length of lamination but create discontinuities in the deck. As a result, longitudinal deck stiffness is decreased, which improves load distribution. However, the discontinuities caused by the butt joints decrease the deck section and reduce load capacity. The decision to use butt joints, and their relative frequency, depends on the availability and relative economics of lumber sizes and must be evaluated on a site-specific basis.

Determine the preliminary lamination layout including the length of laminations and the frequency and location of butt joints. Not more than one butt joint may occur in any four adjacent laminations over a span distance of 4 feet (Figure 9-9).

4. Compute the transverse moduli for the stress-laminated system.

In addition to material design values, stress-laminated deck design must consider the transverse bending modulus, E_{TS}, and the transverse shear modulus, G_{TS} of the stress-laminated system. These values are derived from research data on stress-laminated deck behavior and depend on the species of lumber lamination and the level of prestress (the minimum prestress level required for acceptable deck performance is used). They are based on overall system behavior and should not be confused with the clear wood values discussed in Chapter 3.

At this time, values of E_{TS} and G_{TS} derived by testing are limited to the Douglas Fir-Larch, Hem-Fir (North), Red Pine, or Eastern White Pine laminations. Design values for these species are computed by

$$E_{TS} = 0.013\,E' \tag{9-4}$$

$$G_{TS} = 0.03\,E' \tag{9-5}$$

Research is currently in progress to determine E_{TS} and G_{TS} for other softwoods and hardwoods and values should be available in the near future.

5. Compute maximum vehicle live load moment.

Compute the maximum moment for one wheel line of the design vehicle. Maximum simple-span moments for standard AASHTO vehicles and selected overloads are given in Table 16.8 of Chapter 16. For multiple-span continuous bridges, maximum moments are computed for the controlling truck or lane load by analyzing the deck as a continuous beam.

6. Compute wheel load distribution width.

Stress-laminated decks are designed as a beam, assuming that one wheel line of the design vehicle is distributed over a wheel load distribution width, D_w. The value of D_w is based on orthotropic plate behavior and is slightly larger for decks with butt joints because of the lower longitudinal stiffness caused by the joints. The effect of butt joints on load distribution depends on butt-joint frequency and is expressed by a butt joint factor, C_B, given in Table 9-1.

Determine D_w from Figure 9-15 using values of α and θ computed by

$$\alpha = \frac{2G_{TS}}{\sqrt{E'(C_B)(E_{TS})}} \tag{9-6}$$

$$\theta = \frac{b}{2L}\left[\frac{E'(C_B)}{E_{TS}}\right]^{0.25} \tag{9-7}$$

Table 9-1. - Butt-joint factor, C_B for longitudinal stress-laminated lumber bridges.

Butt joint frequency	C_B
1 in 4	0.80
1 in 5	0.85
1 in 6	0.88
1 in 7	0.90
1 in 8	0.93
1 in 9	0.93
1 in 10	0.94
No butt joints	1.00

Number of butt joints in number of adjacent laminations, measured within a distance of 4 feet along the bridge span (1 in 4 indicates that one butt joint occurs in 4 adjacent laminations as shown in Figure 9-9).

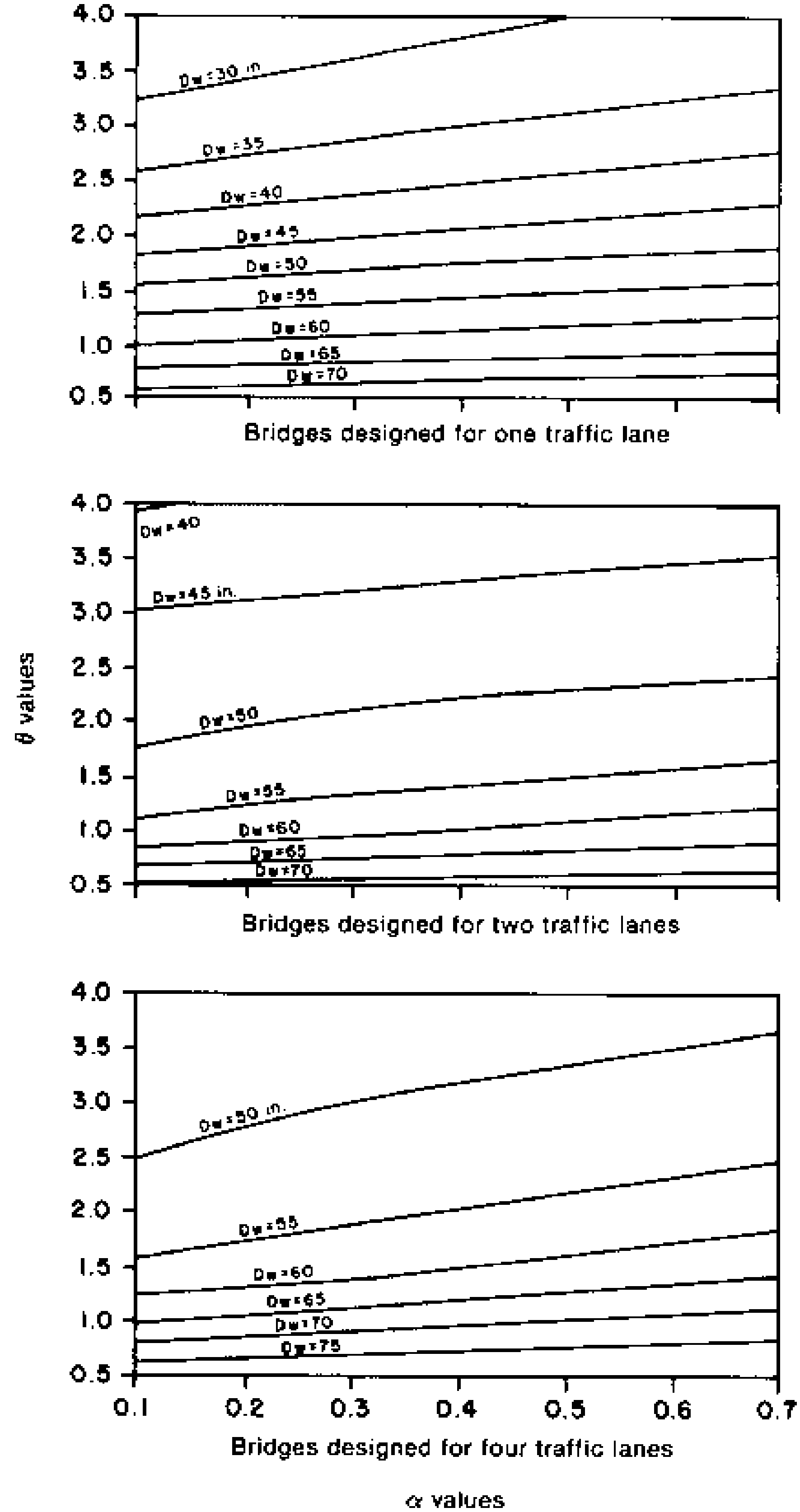


Figure 9-15. - Graphs for determining the wheel load distribution width (D_w) for longitudinal stress-laminated bridge decks.

where G_{TS} = transverse shear modulus of the stress-laminated system (lb/in^2),

E' = allowable modulus of elasticity for the lumber laminations (lb/in^2),

C_B = butt joint factor from Table 9-1,

E_{TS} = transverse modulus of elasticity for the stress-laminated system (lb/in^2),

b = deck width measured between the outside deck edges (ft), and

L = deck span measured center to center of bearings (ft).

The distribution width, D_w, must not be greater than the bridge width divided by the total number of wheel lines, assuming two wheel lines per design traffic lane.

7. Estimate deck thickness and compute effective deck-section properties.

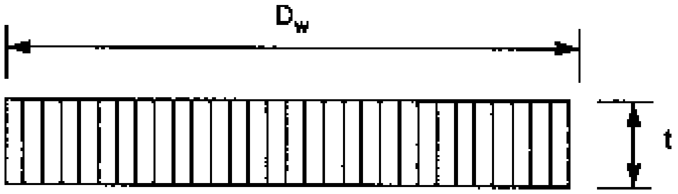
Deck thickness must be estimated for initial calculations. Approximate deck thickness span relationships that may be used for estimating deck thickness are shown in Table 9-2.

Select an initial deck thickness, t, and compute section properties of the effective deck section using Equations 9-8 and 9-9 below (note that D_w is adjusted by C_B). When the hole diameter in laminations for prestressing rods is less than or equal to 20 percent of the deck thickness, holes may be ignored when computing section properties. When the hole diameter exceeds 20 percent of the deck thickness, the hole area must be deducted from the effective deck section.

Table 9-2. - Approximate maximum spans for longitudinal stress-laminated deck bridges for purposes of estimating deck thickness.

Deck thickness (in.)	Approximate maximum span (ft)	
	Single-lane bridges	Double-lane bridges
7-1/4	14	14
8	17	17
9-1/4	22	21
10	24	23
11-1/4	25	24
12	27	26
13-1/4	31	29
14	33	31
15-1/4	37	35
16	39	37

Spans listed in this table are based on HS 20-44 loading and No. 1 Douglas Fir-Larch laminations, and are typically limited by a live load deflection of L/360. Longer spans may be possible with an increased deflection, subject to designer judgment. For lumber species with lower tabulated bending stress or modulus of elasticity, indicated maximum spans should be reduced accordingly.



$$S = \text{effective deck section modulus } \left(\text{in}^3\right) = \frac{D_W(C_B)(t)^2}{6} \tag{9-8}$$

$$I = \text{effective deck moment of inertia } \left(\text{in}^4\right) = \frac{D_W(C_B)(t)^3}{12} \tag{9-9}$$

where t is the deck thickness (in.)

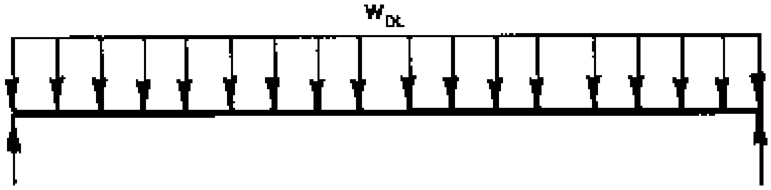
8. Compute deck dead load and dead load moment.

Compute the uniform dead load, DL, of the deck and wearing surface in pounds per square foot using the unit material weights given in Chapter 6. Typical values of DL for decks with asphalt or timber wearing surfaces are given in Table 9-3. From this, determine the uniform dead load acting over D_W per foot of deck span, w_{DL}. When the deck is provided with curbs, railings, or other attached components, the dead load of these components is assumed to be uniformly distributed across the entire deck width and is added to w_{DL}.

Dead load moment for simple-span decks with uniform loads is computed by Equation 9-10:

Table 9-3. -Typical dead load unit weights for stress-laminated lumber bridge decks.

	Dead load (lb/ft²)		
Deck thickness (in.)	Deck only	Deck with 3-inch asphalt surface	Deck with 3-inch lumber surface
7-1/4	30.2	67.7	42.7
8	33.3	70.8	45.8
9-1/4	38.5	76.0	51.0
10	41.7	79.2	54.2
11-1/4	46.9	84.4	59.4
12	50.0	87.5	62.5
13-1/4	55.2	92.7	67.7
14	58.3	95.8	70.8
15-1/4	63.5	101.0	76.0
16	66.7	104.2	79.2



$$M_{DL} = \frac{w_{DL}L^2}{8} \tag{9-10}$$

where M_{DL} = maximum dead load moment (ft-lb),

w_{DL} = uniform dead load over the wheel load distribution width, D_w, per foot of deck span (lb/ft), and

L = bridge span length (ft).

9. Compute bending stress.

Deck bending stress is computed by dividing the sum of the maximum live load and dead load bending moments by the effective deck section modulus, as computed by

$$f_b = \frac{M}{S} \tag{9-11}$$

where $M = M_{DL} + M_{LL}$, the sum of the maximum dead load moment and the maximum live load moment from one wheel line of the design vehicle (in-lb), and

S = effective deck section modulus from Equation 9-8 (in^3).

The applied bending stress must not exceed the allowable bending stress for the selected species and grade of lumber lamination, as computed by

$$f_b \le F_b' \tag{9-12}$$

The allowable bending stress may be increased by a factor of 1.33 for overloads in AASHTO Load Group IB.

If $f_b \le F_b'$, the deck is sufficient in bending. If f_b is substantially less than F_b', a thinner deck or lower-grade material may be more economical; however, no changes in deck thickness or grade should be made until after live load deflection is checked.

If $f_b > F_b'$, the deck is insufficient in bending and the initial deck thickness or lumber grade (tabulated bending stress) must be increased. In either case, the design sequence must be repeated.

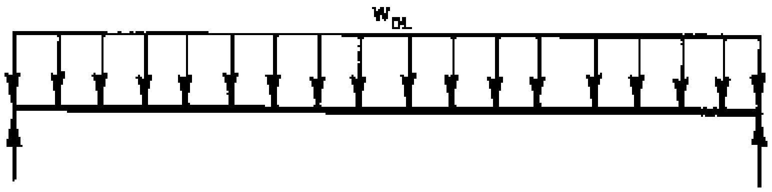
10. Check live load deflection.

Live load deflection is computed by standard methods of elastic analysis for one wheel line of the design vehicle. Because deflection is a serviceability design criterion, an acceptable method without safety factors is desired. Because the orthotropic behavior of the deck results in a wider distribution width for deflection than for bending, the deck moment of inertia used to calculate live load deflection should be taken as 1.33 times the effective deck moment of inertia computed by Equation 9-9. Deflection coefficients for standard AASHTO loads and selected overloads on simple spans are given in Table 16-8. The computed live load deflection must not exceed the allowable deflection established for the structure. If deflection exceeds an acceptable level, the deck thickness or modulus of elasticity must be increased and the design sequence repeated.

Recommended live load deflection criteria for timber bridges is not specified by AASHTO, and the maximum permissible deflection is left to designer judgment. The maximum live load deflection recommended for a stress-laminated deck with asphalt wearing surface is $L/360$. If the structure is provided with a pedestrian walkway, a further reduction in live load deflection is recommended to avoid dynamic effects and the human perception of motion. Acceptance of deflection values exceeding $L/360$ is at the designers discretion and should be related to the relative magnitude of the deflection and its effect on the overall bridge performance.

11. Compute dead load deflection and camber.

For longitudinal stress-laminated lumber decks with butt joints, it is recommended that the bridge be cambered to offset sagging caused by long-term creep. The amount of camber depends on the initial dead load deflection resulting from the uniform dead load acting over a deck width, D_w. For a simple-span deck, dead load deflection is computed by Equation 9-13:



$$\Delta_{DL} = \frac{5 w_{DL} L^4}{384\, E' I} \qquad (9\text{-}13)$$

where Δ_{DL} = dead load deflection (in.),

w_{DL} = uniform dead load over the wheel load distribution width, D_w, per inch of deck span (lb/in),

L = deck span (in.), and

I = effective deck moment of inertia from Equation 9-9 (in^4).

The amount of camber placed in the deck at the time of stressing should be a minimum of two times, and preferably three times, the computed deck dead load deflection.

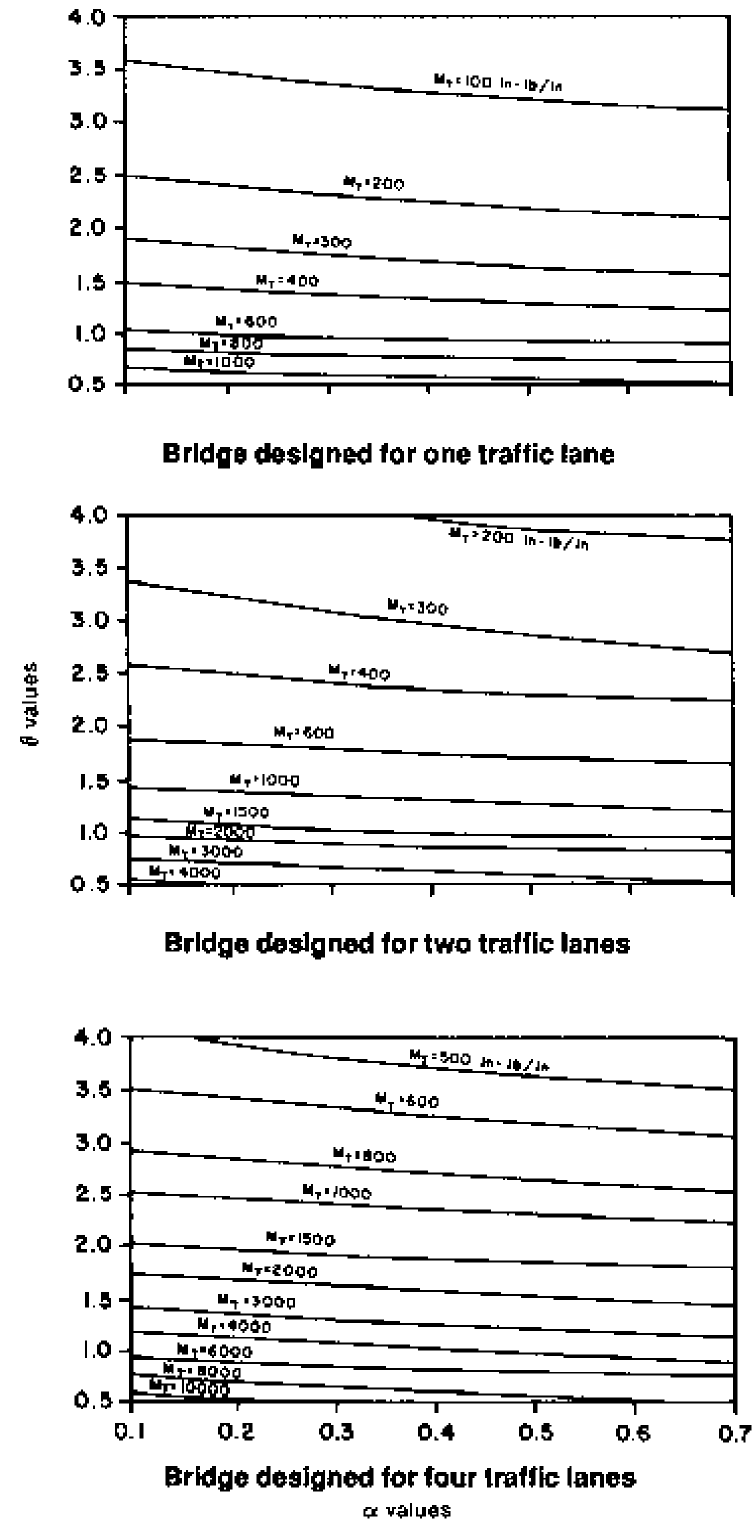
12. Determine the required prestress level.

The level of compressive prestress between the laminations must be sufficient to offset flexural tension stress caused by transverse moment and slip caused by transverse shear. For stress-laminated deck design, the level of uniform prestress must be determined for two conditions; in service and at installation. The prestress level in service represents the minimum compressive prestress required for adequate deck performance, assuming all time-related stress loss has occurred. The prestress level at installation is the amount of prestress that must be introduced into the deck at the time of stressing.

Required compressive prestress levels depend on the magnitude of transverse bending and transverse shear from applied loads. Values for both forces are determined from curves based on orthotropic deck behavior in response to applied wheel loads. The magnitude of transverse bending moment, M_T, is obtained from Figure 9-16 using the values of α and θ computed by Equations 9-6 and 9-7. Transverse shear, V_T, is determined from Figure 9-17 using the parameter β, defined by

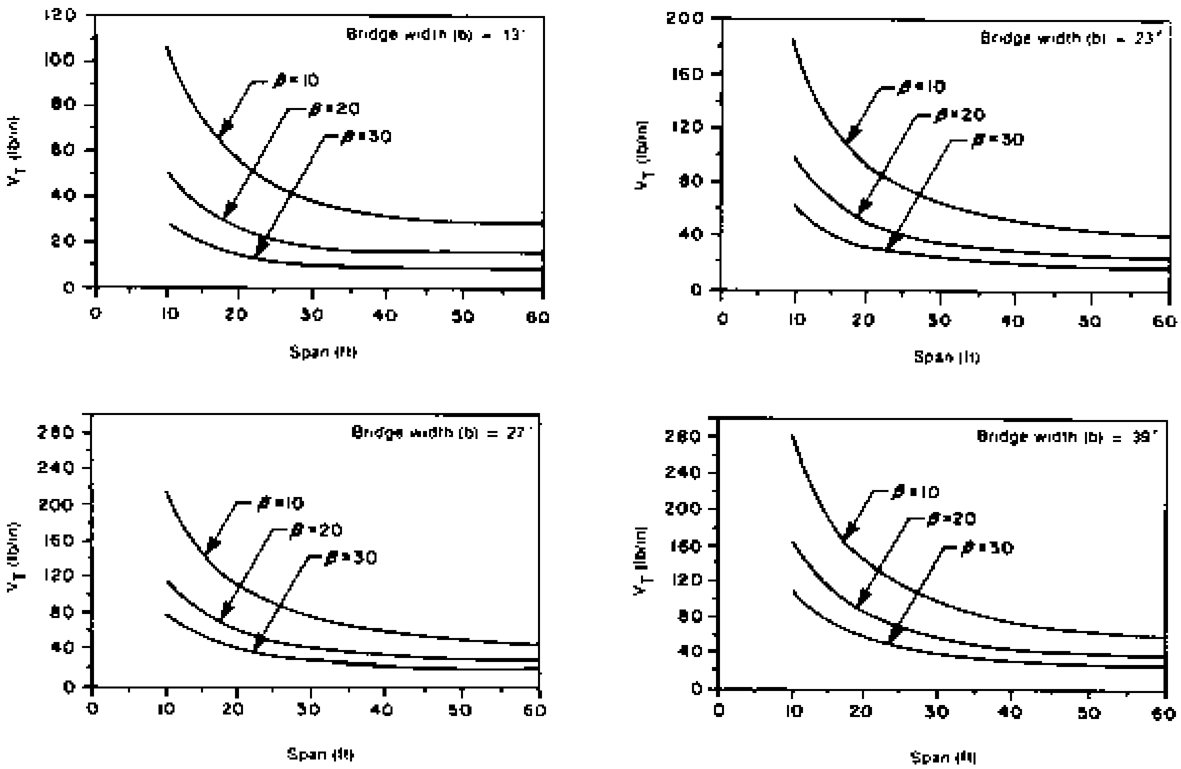
$$\beta = \pi\left(\frac{b}{L}\right)\sqrt{\frac{E'(C_B)}{2G_{TS}}} \qquad (9\text{-}14)$$

Values of M_T and V_T obtained from Figures 9-16 and 9-17 are based on an HS 20-44 truck with a 16,000-pound wheel load. When other wheel loads



Graphs are based on a HS 20-44 vehicle with maximum wheel load of 16,000-lb. For other wheel loads, multiply the graph value of M_t by the ratio of design wheel load to a 16,000-lb wheel load.

Figure 9-16. - Graphs for determining the magnitude of transverse bending (M_T) for longitudinal stress-laminated bridge decks.



Graphs are based on a HS 20-44 vehicle with a maximum wheel load of 16,000 lb. For other wheel loads, multiply the graph value of V_T by the ratio of design wheel load to a 16,000-lb wheel load. Use interpolation and/or extrapolation for intermediate bridge widths and β values.

Figure 9-17. - Graphs for determining the magnitude of transverse shear (V_T) for longitudinalstress-laminated bridge decks.

are used, values must by multiplied by the ratio of the design wheel load to the HS 20-44 wheel load.

The minimum level of uniform compressive prestress in service, *N*, is the largest value obtained from the following equations, but not less than 40 lb/in², as computed by

$$N = \frac{6M_T}{t^2} \quad \text{or} \quad N = \frac{1.5V_T}{t(\mu)}\text{, whichever is greater} \tag{9-15}$$

$$N \geq 40 \text{ lb/in}^2 \tag{9-16}$$

where *N* = minimum uniform compressive prestress in service (lb/in²),

t = deck thickness (in.),

M_T = magnitude of transverse bending from applied wheel loads (in-lb/in),

V_T = magnitude of transverse shear from applied wheel loads (lb/in), and

μ = coefficient of friction (0.35 for surfaced (S4S) lumber, 0.45 for rough-sawn lumber or lumber that is surfaced on one side (S1S)).

Over the bridge life, time-related creep losses are assumed to reduce the level of compressive prestress to 40 percent of the initial level at installation 60-percent stress loss). This assumption is based on research and field performance for softwood laminations that are properly treated with oil-type preservatives and are installed at a moisture content of 19 percent or less (there has been no research or experience with hardwood species or waterborne treatments; however, research in these areas is in progress). To compensate for the gradual 60-percent stress loss, the level of uniform prestress at the time of installation, N_i, must be greater than or equal to 2.5 times the minimum required prestress level in-service, as computed by

$$N_i \geq 2.5N \tag{9-17}$$

where N_i is the level of uniform compressive prestress required at the time of installation (lb/in^2).

13. Determine spacing and size of prestressing rods and the required prestressing force.

Prestressing rods for stress-laminated decks are threaded high-strength steel conforming to ASTM A 722, Uncoated High-Strength Steel Bar for Prestressing Concrete. The rods are 5/8-inch, 1-inch or 1-1/4-inch diameter with properties shown in Table 9-4. The specified minimum ultimate tensile stress of the prestressing rods, f_{pu}, is 150,000 lb/in^2. The maximum allowable tensile stress, at or after anchorage, cannot exceed 70 percent of ultimate tensile strength (105,000 lb/in^2). During jacking, the maximum short-term tensile stress cannot exceed 80 percent of the ultimate tensile strength (120,000 lb/in^2). These values should be further reduced by any strength reductions recommended by the rod manufacturer.

The spacing of the prestressing rods, S_p, must be sufficient to induce the required uniform compressive prestress in areas adjacent to vehicle wheel loads. As previously discussed, compressive prestress is not uniform at the deck edge but becomes uniform at some interior distance. Rod spacing therefore depends on the wheel load placement in relation to the deck edge. Using the same requirements previously discussed for other timber deck systems, the center of the wheel load is placed 1 foot from the nearest face of the curb or rail.

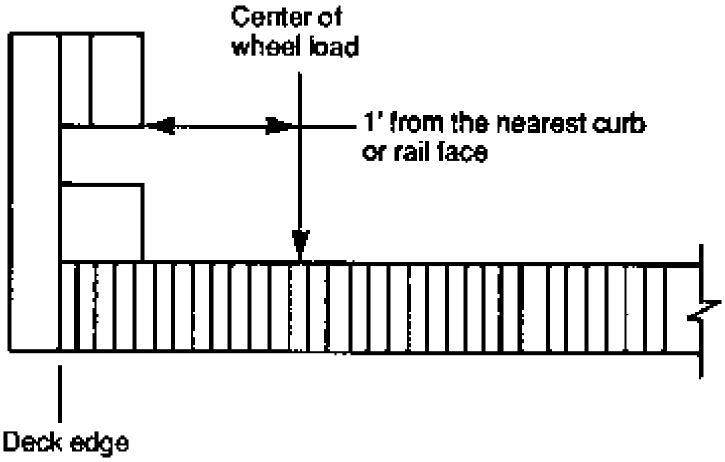
Table 9-4. - Properties of steel prestressing rods used for stress-laminated lumber bridge decks.

Rod diameter (in.)	Rod area, A_S (in^2)	Maximum allowable tensile load[a] (lb) At or after anchorage[b]	During jacking (short-term)[c]
5/8	0.28	29,400	33,600
1	0.85	89,250	102,000
1-1/4	1.25	131,250	150,000

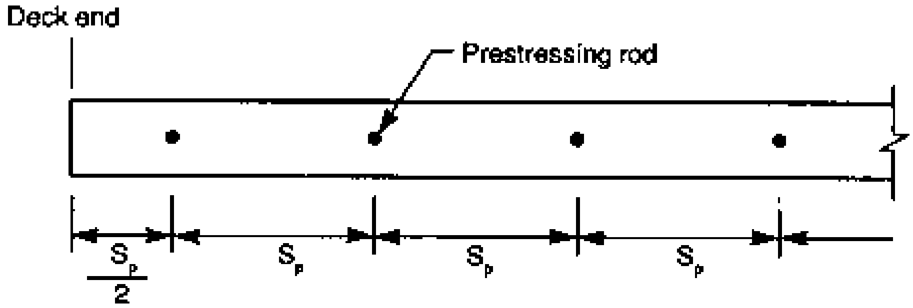
[a] For rods conforming to ASTM A 722 with a specified minimum ultimate tensile strength, f_{pu}, of 150,000 lb/in^2.

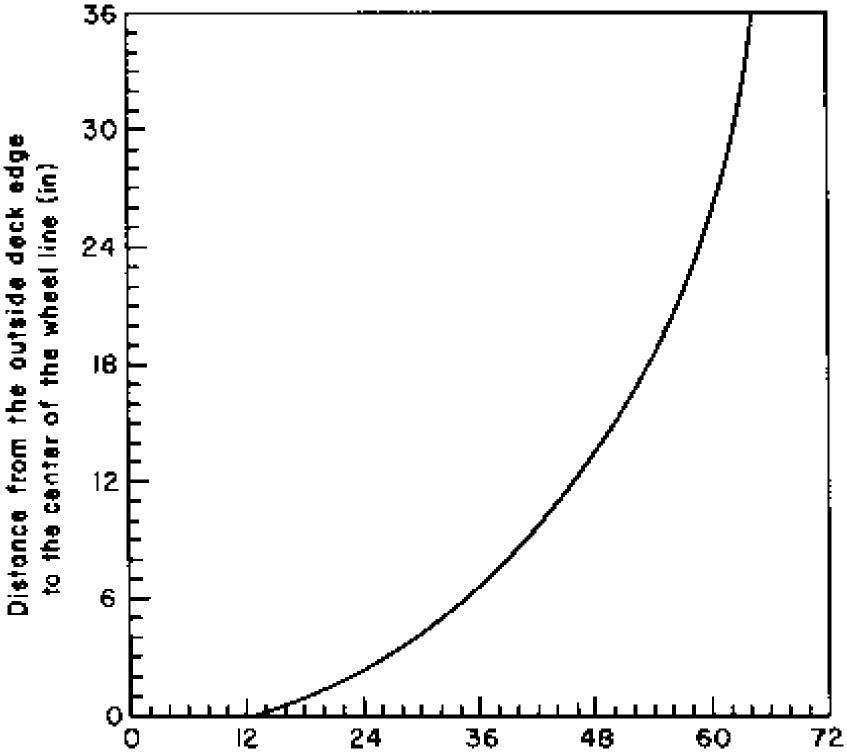
[b] $0.70 f_{pu} A_S$

[c] $0.80 f_{pu} A_S$



The maximum spacing of prestressing rods is obtained using the curve in Figure 9-18, based on the distance from the outside deck edge to the center of the wheel load. The spacing of the first rod from the deck end is generally equal to one-half the center-to-center spacing.





Maximum c-c spacing of prestressing rods (in)

Figure 9-18. - Maximum spacing of prestressing rods as a function of the distance from the outside deck edge to the center of the vehicle wheel line.

The size of the prestressing rods depends on the required level of compressive prestress at installation, N_i, and the rod spacing, S_p. In addition, rod area must be limited so that the ratio of the steel area to the wood area is less than or equal to 0.0016, as computed by

$$\frac{N_i(S_P)(t)}{0.70(f_{pu})} \leq A_S \qquad \text{and} \qquad \frac{A_s}{S_P(t)} < 0.0016 \qquad (9\text{-}18)$$

where A_S = cross-sectional area of the steel prestressing rod (in^2),

N_i = level of uniform prestress required at the time of installation (lb/in^2),

S_P = center-to-center spacing of the prestressing rods (in.),

t = deck thickness (in.), and

f_{pu} = specified minimum ultimate tensile stress for the prestressing rod, 150,000 lb/in^2.

Select a rod spacing and diameter that satisfy maximum spacing requirements and the steel area requirements of Equation 9-18. Rod spacing should also consider possible conflicts with other structural components, such as guardrail posts. Approximate spacing requirements for various rod diameters and deck thicknesses are given in Table 9-5.

The prestressing force required in each prestressing rod, F_{ps}, is computed by

$$F_{ps} = N_i(S_p)(t) \tag{9-19}$$

14. Design the anchorage system.

The anchorage system for prestressing rods must securely hold the rods and effectively transfer the prestressing force to the lumber laminations. In addition, the anchorage must be of sufficient size to prevent excessive wood crushing in the outside laminations. The two anchorage configurations used are the bearing-plate configuration developed at UW/FPL and the channel bulkhead configuration developed in Ontario and included in the OHBDC. With the exception of the high-strength steel rods and nuts, components for both systems are normally fabricated of galvanized steel (ASTM A 36) or weathering steel (ASTM A 588).

As previously discussed, the bearing-plate anchorage configuration may result in some localized wood crushing in the vicinity of the bearing plates that may not be acceptable in all cases. The channel bulkhead configuration covers the outside laminations with a steel channel and any wood crushing

Table 9-5. - Approximate spacing requirements for prestressing rods used for stress-laminated lumber decks.

	Rod spacing (in.)					
	5/8-in. ø rods		1-in. ø rods		1-1/4 in. ø rods	
t (in.)	Max.	Min.	Max.	Min.	Max.	Min.
7-1/4	41	24	—	—	—	—
8	37	22	—	—	—	—
9-1/4	32	19	—	—	—	—
10	29	18	89	53	—	—
11-1/4	26	16	79	47	—	—
12	25	15	74	44	—	—
13-1/4	—	—	67	40	99	59
14	—	—	64	38	94	56
15-1/4	—	—	59	35	86	51
16	—	—	56	33	82	49

Maximum rod spacing is based on a uniform compressive prestress level of 100 lb/in^2.
Minimum rod spacing is based on a maximum wood/steel ratio of 0.0016.

is not visible; however, the channel bulkhead is more costly. Design procedures for both configurations are presented below. The choice of the most appropriate system is left to designer judgment based on specific project requirements.

Bearing-Plate Anchorage Configuration

The bearing-plate anchorage consists of an inner-steel bearing plate, an outer-steel anchorage plate and a high-strength steel nut (Figure 9-19). Design of this anchorage primarily involves determining the length, width, and thickness of the inner bearing plate. The outer anchorage plate is available from the rod manufacturer and is normally standardized (by manufacturer) based on the prestressing rod diameter (Table 9-6).

The area of the bearing plate must be sufficient to limit compressive stress under the plate to the allowable compression perpendicular to grain for the lumber laminations, as computed by

$$A_p \geq \frac{F_{ps}}{F_{c\perp}'} \tag{9-20}$$

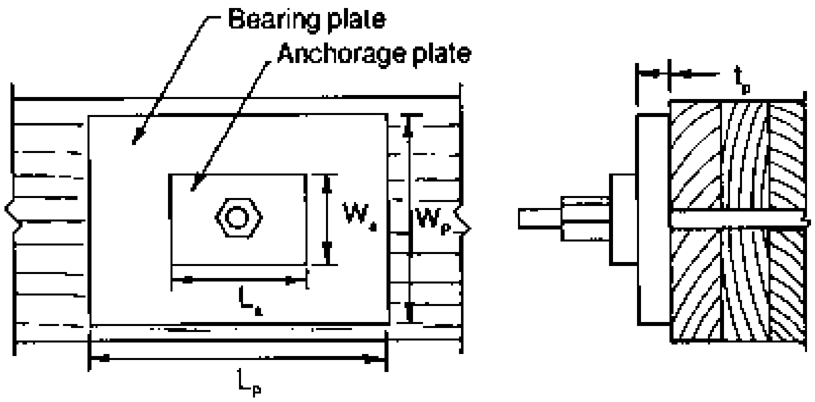


Figure 9-19. - Bearing-plate anchorage configuration.

Table 9-6. - Typical sizes for prestressing-rod anchorage plates.

Prestressing rod diameter (in.)	Anchorage plate dimensions (in.) width (W_A) x length (L_A) x thickness (t_A)	
	Typical plate size	**Alternate plate size**
5/8	2 x 5 x 1	3 x 3 x 0.75
1	4 x 6.5 x 1.25	4 x 7 x 1
1-1/4	5 x 8 x 1.5	5 x 8 x 1.25

Plate sizes may vary and should be verified with the rod manufacturer. Other sizes may be specified by the designer to meet specific design requirements.

where A_p = bearing plate area (in^2),

F_{ps} = rod prestressing force, from Equation 9-19 (lb), and

$F_{c\perp}'$ = allowable stress in compression perpendicular to grain for the lumber laminations (lb/in^2).

In addition, the ratio of the bearing plate length to width must not be less than 1.0, nor greater than 2.0, as computed by

$$1.0 \geq \frac{L_p}{W_p} \geq 2.0 \tag{9-21}$$

where L_p = bearing-plate length (in.), and

W_p = bearing-plate width (in.).

Determine an acceptable bearing plate size based on the requirements of Equations 9-20 and 9-21 and compute the lamination bearing stress in compression perpendicular to grain by

$$f_{c\perp} = \frac{F_{ps}}{A_p} \tag{9-22}$$

where $f_{c\perp}$ is the applied bearing stress in compression perpendicular to grain (lb/in^2).

Based on the bearing-plate area and bearing stress, select a bearing-plate thickness that satisfies:

$$t_p = \sqrt{\frac{3(f_{c\perp})(k^2)}{F_b}} \tag{9-23}$$

where

$$k = \frac{W_p - W_A}{2} \quad \text{or} \quad k = \frac{L_p - L_A}{2}\text{, whichever is greater} \tag{9-24}$$

t_p = bearing plate thickness (in.),

F_b = $0.55F_y$ = allowable bending stress for the steel plate (lb/in^2),

F_y = specified minimum yield point for the steel plate (lb/in^2), from AASHTO Table 10.2A (36,000 lb/in^2 for A36 steel and 50,000 lb/in^2 for A588 steel),

W_A = anchor-plate width (in.), and

L_A = anchor-plate length (in.).

If an acceptable bearing-plate size that limits compression perpendicular to grain to an allowable value cannot be achieved, or if the plate thickness is excessive, rod spacing must be decreased, and the anchorage design must be repeated.

Channel Bulkhead Anchorage Configuration

The channel bulkhead anchorage consists of a continuous steel channel, a steel bearing plate, and a high-strength steel nut (Figure 9-20). For this anchorage configuration, design involves sizing both the steel channel and the bearing plate. The design provisions given here are based on current requirements of OHBDC.

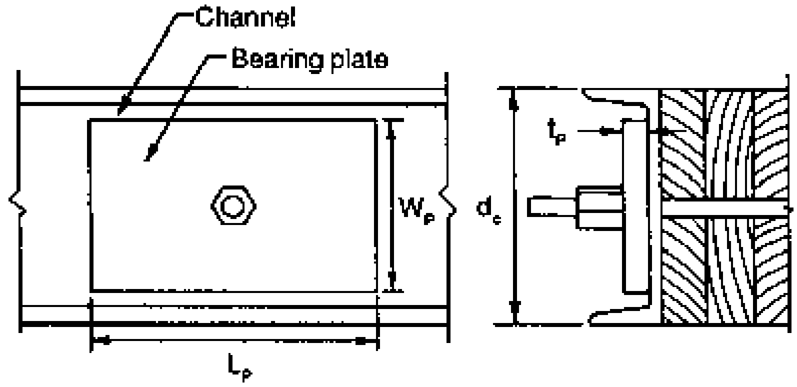


Figure 9-20. - Channel bulkhead anchorage configuration.

The steel channel for the bulkhead configuration is continuous along the bridge span but may be discontinuous over supports. Channel depth is based on deck thickness and must be within 85 and 100 percent of the lamination depth, as computed by

$$0.85t \leq d_c \leq t \tag{9-25}$$

where t = deck thickness (in.), and

d_c = depth of steel channel (in.).

Section properties for steel channels should also meet minimum requirements given in Table 9-7. Select a channel size based on the requirements of Equation 9-25 and Table 9-7 and compute an initial bearing-plate length using

$$L_p \geq \frac{F_{ps}}{d_c F_{c\perp}'} - 2(t_W) \tag{9-26}$$

where: L_p = bearing-plate length (in.),

F_{ps} = rod prestressing force, from Equation 9-19 (lb),

Table 9-7. - Minimum section properties for steel channel bulkheads used for stress-laminated lumber decks.

Nominal lamination depth, t (in.)	Minimum channel moment of inertia[a] (in^2)	Minimum channel web thickness (in.)
8	1.3	0.38
10	2.4	0.43
12	3.3	0.43
14	5.1	0.51
16	9.2	0.52

[a] Moment of inertia about the minor axis.

$F_{c\perp}'$ = allowable stress in compression perpendicular to grain for the lumber laminations (lb/in^2), and

t_W = steel channel web thickness (in.).

Select the bearing-plate width and thickness based on (bearing-plate width must also permit the plate to fit between the tapered flanges of the channel)

$$1.0 \leq \frac{L_P}{W_P} \leq 2.0 \tag{9-27}$$

$$t_P \geq \frac{L_P}{12} \tag{9-28}$$

where W_P = bearing-plate width (in.), and

t_P = bearing-plate thickness (in.).

The bearing area of the channel bulkhead must be sufficient to limit the compressive stress at the anchorage to the allowable compressive stress perpendicular to grain for the lumber laminations. The effective bearing area, A_E, is based on a length equal to the bearing-plate length plus twice the channel thickness, and a width equal to the channel depth, as computed by

$$A_E = d_c(L_P + 2t_W) \tag{9-29}$$

where A_E is the effective bearing area in in^2.

The bearing stress in compression perpendicular to grain is computed by

$$f_{c\perp} = \frac{F_{ps}}{A_E} \tag{9-30}$$

This value must not exceed the allowable compression perpendicular to grain for the lumber laminations computed by

$$f_{c\perp} \leq F_{c\perp}' \tag{9-31}$$

If $f_{c\perp} > F_{c\perp}'$, the size of the bearing plate or steel channel must be increased or the rod spacing must be decreased. In either case, the anchorage design must be repeated.

15. Determine the support configuration and check bearing stress.

Support attachments for longitudinal stress-laminated decks must be designed to resist the vertical and lateral forces transmitted from the superstructure to the substructure. As with other longitudinal deck superstructures, the required bearing length is normally controlled by considerations for bearing configuration, rather than stress in compression perpendicular to grain. From a practical standpoint, a bearing length of 10 to 12 inches is recommended for stress-laminated decks. Bearing attachments are normally made through the deck to the supporting cap or sill, or from the deck underside, using the same details previously discussed for longitudinal glulam decks (Figure 8-7).

Stress in compression perpendicular to grain at the bearing is checked for a deck width equal to the wheel load distribution width, D_w, using

$$f_{c\perp} = \frac{R_{DL} + R_{LL}}{D_w(\ell_b)} \tag{9-32}$$

where R_{DL} = dead load reaction for a deck width D_w, based on the out-out bridge length (lb),

R_{LL} = maximum reaction produced by one wheel line of the design vehicle, from Table 16-8 (lb), and

ℓ_b = bearing length (in.).

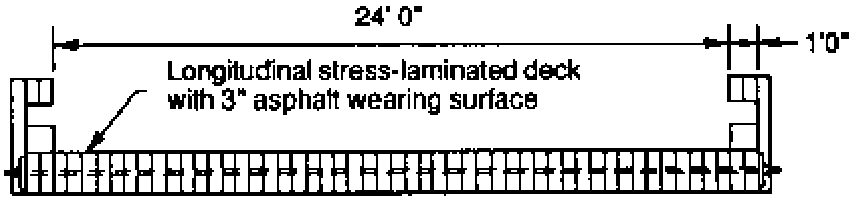
Stress in compression perpendicular to grain must not exceed the allowable stress for the species and grade of lumber lamination, as computed by

$$f_{c\perp} \leq F_{c\perp}' \tag{9-33}$$

Example 9-1 - Longitudinal stress-laminated lumber deck; two-lane, HS 20-44 loading

An existing bridge on a county road will be replaced with a longitudinal stress-laminated lumber deck bridge. The bridge spans 37 feet center-to-center of bearings and carries two lanes of AASHTO HS 20-44 on a roadway width of 24 feet. Support for the structure is provided by existing pile abutments with 12-inch-wide caps. Design this bridge, assuming the following:

1. The bridge will include 12-inch by 12-inch timber curbs and vehicular railing with a combined dead load of 85 lb/ft, per side.
2. The deck will be paved with 3 inches of asphalt pavement.
3. Live load deflection must be limited to *L/360.*
4. Lumber laminations are full-sawn, surfaced one side (S1S) Douglas Fir-Larch.
5. The deck will have butt joints at the minimum spacing.
6. A bearing plate anchorage configuration will be used.



Solution

Define Deck Geometric Requirements and Design Loads

The bridge supports two traffic lanes over a span of 37 feet. With a road-way width of 24 feet, and 12-inch-wide curbs on each side, a bridge width of 26 feet is required. Design loading will be one HS 20-44 wheel line in AASHTO Load Group I.

Select a Species and Grade of Lamination and Compute Allowable Design Values

From the NDS Table 4A, Douglas Fir-Larch that is visually graded No. 1 or better in the J&P size classification is selected. Tabulated values are as follows:

F_b = 1,500 lb/in^2 (single-member use)

E = 1,800,000 lb/in^2

$F_{c\perp}$ = 625 lb/in^2

Allowable design values are computed using the applicable moisture content factor (C_M) from Table 5-7. The tabulated bending stress for single-member use is increased by the load sharing factor, $C_{LS} = 1.50$:

$$F_b' = F_b C_M C_{LS} = 1{,}500(0.86)(1.50) = 1{,}935 \text{ lb/in}^2$$

$$E' = EC_M = 1{,}800{,}000(0.97) = 1{,}746{,}000 \text{ lb/in}^2$$

$$F_{c\perp}' = F_{c\perp} C_M = 625(0.67) = 419 \text{ lb/in}^2$$

Determine the Preliminary Lamination Layout

The minimum butt joint spacing is assumed. Not more than one butt joint will occur in any four adjacent laminations within a span distance of 4 feet.

Compute the Transverse Moduli for the Stress-Laminated System

Values of the transverse bending modulus (E_{TS}) and transverse shear modulus (G_{TS}) are computed by Equations 9-4 and 9-5:

$$E_{TS} = 0.013E' = 0.013(1{,}746{,}000) = 22{,}698 \text{ lb/in}^2$$

$$G_{TS} = 0.03E' = 0.03(1{,}746{,}000) = 52{,}380 \text{ lb/in}^2$$

Compute Maximum Live Load Moment

The maximum live load moment for one wheel line of an HS 20-44 truck on a 37-foot span is obtained from Table 16-8:

$$M_{LL} = 198{,}300 \text{ ft-lb}$$

Compute Wheel Load Distribution Width

Values of α and θ are computed using Equations 9-6 and 9-7, respectively. Assuming one butt joint in every 4 adjacent laminations, a butt joint factor $C_B = 0.80$ is obtained from Table 9-1.

$$\alpha = \frac{2G_{TS}}{\sqrt{E'(C_B)E_{TS}}} = \frac{2(52{,}380)}{\sqrt{1{,}746{,}000(0.80)(22{,}698)}} = 0.59$$

$$\theta = \frac{b}{2L}\left[\frac{E'(C_B)}{E_{TS}}\right]^{0.25} = \frac{26}{2(37)}\left[\frac{1{,}746{,}000(0.80)}{22{,}698}\right]^{0.25} = 0.98$$

The distribution width, D_W, is obtained from Figure 9-15 using the curves for bridges with two traffic lanes:

$$D_W = 63 \text{ in.}$$

Estimate Deck Thickness and Compute Effective Section Properties

From Table 9-2, an initial nominal deck thickness of 16 inches is selected. Effective deck section properties are computed by Equations 9-8 and 9-9, assuming that holes for the prestressing rods are less than 20 percent of the deck thickness:

$$S = \frac{D_w\,(C_s)\,(t)^2}{6} = \frac{63\,(0.80)\,(16)^2}{6} = 2{,}150 \text{ in}^3$$

$$I = \frac{D_w\,(C_s)\,(t)^3}{12} = \frac{63\,(0.80)\,(16)^3}{12} = 17{,}203 \text{ in}^4$$

Compute Deck Dead Load and Dead Load Moment

From Table 9-3, the dead load of the 16-inch deck with a 3-inch asphalt wearing surface is 104.2 lb/ft^2 The 85lb/ft dead load for the curb and railing is increased by an estimated 10 lb/ft for the prestressing system, and is assumed to be uniformly distributed across the deck width:

$$\text{DL} = 104.2 \text{ lb/ft}^2 + \frac{2(\,85 \text{ lb/ft} + 10 \text{ lb/ft}\,)}{26 \text{ ft}} = 111.5 \text{ lb/ft}^2$$

For the distribution width of 63 in.,

$$w_{DL} = \frac{63 \text{ in.}}{12 \text{ in/ft}}\left(111.5 \text{ lb/ft}^2\right) = 585.4 \text{ lb/ft}$$

Maximum dead load moment is computed by Equation 9-10:

$$M_{DL} = \frac{w_{DL}L^2}{8} = \frac{585.4(37)^2}{8} = 100{,}117 \text{ ft-lb}$$

Compute Bending Stress

Bending stress is computed by Equation 9-11:

$$f_b = \frac{M}{S} = \frac{(198{,}300 + 100{,}177)(12 \text{ in/ft})}{2{,}150 \text{ in}^3} = 1{,}666 \text{ lb/in}^2$$

f_b = 1,666 lb/in^2 < F_b' = 1,935 lb/in^2, so bending stress is acceptable. Because of the large difference between f_b and F_b', it may be possible to reduce deck thickness, but no changes will be made until after deflection is checked.

Check Live Load Deflection

From Table 16-8, the deflection coefficient for one wheel line of an HS 20-44 truck on a 37-foot simple span is 4.74 x 10^{10} lb-in^3. Live load deflection is computed using 133 percent of the effective deck moment of inertia:

$$\Delta_{LL} = \frac{4.74 \times 10^{10}}{E'(1.33)(I)} = \frac{4.74 \times 10^{10}}{1{,}746{,}000(1.33)(17{,}203)} = 1.19 \text{ in.} = L/373$$

$L/373 < L/360$, so live load deflection is satisfactory. The deflection is close to the allowable level, so a reduction in deck thickness is not feasible.

Compute Dead Load Deflection and Camber

Dead load deflection is computed by Equation 9-13 for w_{DL} = 585.4 lb/ft:

$$\Delta_{DL} = \frac{5w_{DL}L^4}{384E'I} = \frac{5(585.4)[37\ (12\ \text{in/ft})]^4}{(12\ \text{in/ft})(384)(1{,}746{,}000)(17{,}203)} = 0.82\ \text{in.}$$

The deck will be cambered a minimum of 2.5 inches, which is approximately 3 times the computed dead load deflection.

Determine the Required Prestress Level

Using the previously computed values of α and θ, M_T is obtained for a two-lane bridge from Figure 9-16:

M_T = 1,500 in-lb/in

The variable β is computed by Equation 9-14:

$$\beta = \pi\left(\frac{b}{L}\right)\sqrt{\frac{E'(C_b)}{2G_{TS}}} = 3.14\left(\frac{26}{37}\right)\sqrt{\frac{1{,}746{,}000\ (0.80)}{2\ (52{,}380)}} = 8.06$$

By interpolation and extrapolation of Figure 9-17,

V_T = 80 lb/in

The minimum required level of compressive prestress in service, N, is the largest value computed by Equation 9-15, but not less than 40 lb/in^2:

$$N = \frac{6M_T}{t^2} = \frac{6(1{,}500)}{(16)^2} = 35.2\ \text{lb/in}^2$$

Based on transverse shear,

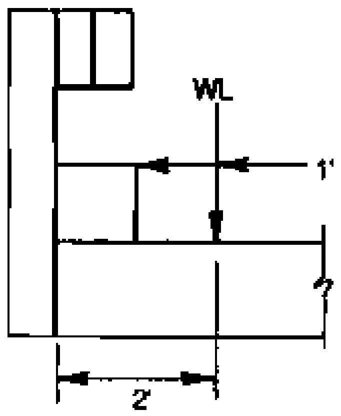
$$N = \frac{1.5V_T}{t(\mu)} = \frac{1.5(80)}{16(0.45)} = 16.7\ \text{lb/in}^2$$

Both values are less than the minimum 40 lb/in^2, so N = 40 lb/in^2 will control. Based on this value, the required level of uniform prestress at installation, N_i, is computed by Equation 9-17:

$N_i = 2.5N = 2.5(40) = 100$ lb/in^2

Determine Spacing and Size of Prestressing Rods and the Required Prestressing Force

Positioning the wheel line 1 foot from the curb face places the center of the wheel line 2 feet from the deck edge:



Using the curve in Figure 9-18, the maximum spacing of prestressing rods is approximately 58 inches.

From Table 9-5, 1-inch-diameter ASTM A722 rods are selected. For the 16-inch deck thickness, rods must be spaced between 33 and 56 inches on-center. For a bridge length of 38 feet (37-foot span on 1-foot-wide sills), a spacing of 48 inches will be used, with end rods 12 inches from the deck end:

From Table 9-4 for a 1-inch-diameter rod, A_S = 0.85 in^2. The minimum required rod area and the steel/wood ratio are checked by Equation 9-18:

$$\frac{N_i S_p t}{0.70 f_{pu}} = \frac{100(48)(16)}{0.70(150{,}000)} = 0.73 \text{ in}^2 < 0.85 \text{ in}^2$$

$$\frac{A_S}{S_p(t)} = \frac{0.85}{48(16)} = 0.0011 < 0.0016$$

The prestressing force required in each rod, F_{ps}, is computed by Equation 9-19:

$$F_{ps} = N_i(S_p)(t) = 100(48)(16) = 76{,}800 \text{ lb}$$

Design Anchorage System

Using the bearing plate anchorage configuration illustrated in Figure 9-19, the minimum bearing plate area is computed by Equation 9-20:

$$A_p = \frac{F_{ps}}{F_{c\perp}'} = \frac{76{,}800}{419} = 183 \text{ in}^2$$

For the 16-inch-thick deck, a plate depth, W_p, of 14 inches is selected. The minimum required plate length is computed by dividing the plate area by the plate width:

$$L_p = \frac{A_p}{W_p} = \frac{183}{14} = 13.1 \text{ in.}$$

A 14-inch length will be used, and

$W_p = 14$ in.

$L_p = 14$ in.

$A_p = (14 \text{ in.})(14 \text{ in.}) = 196 \text{ in}^2$

The ratio of the bearing plate length to width is checked by Equation 9-21:

$$\frac{L_p}{W_p} = \frac{14}{14} = 1.0$$

$1.0 \leq 1.0 \leq 2.0$, so plate dimensions are satisfactory.

Bearing stress in compression perpendicular to grain is computed by Equation 9-22:

$$f_{c\perp} = \frac{F_{ps}}{A_p} = \frac{76{,}800}{196} = 392 \text{ lb/in}^2$$

$f_{c\perp} = 392 \text{ lb/in}^2 < F_{c\perp}' = 419 \text{ lb/in}^2$, so bearing stress is acceptable.

Dimensions for the steel anchorage plate are obtained from Table 9-6 and *k* values are computed by Equation 9-24:

$W_A = 4$ in.

$L_A = 6.5$ in.

$t_A = 1.25$ in.

$$k = \frac{W_P - W_A}{2} = \frac{14 - 4}{2} = 5.00 \text{ in.}$$

or

$$k = \frac{L_P - L_A}{2} = \frac{14 - 6.5}{2} = 3.75 \text{ in.}$$

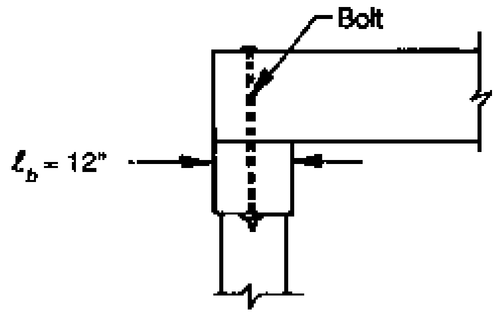
The largest *k* value of 5.00 controls and the required bearing plate thickness for an A36 steel plate is computed by Equation 9-23:

$$t_P = \sqrt{\frac{3(f_{c\perp})(k^2)}{F_b}} = \sqrt{\frac{3(392)(5)^2}{0.55(36{,}000)}} = 1.2 \text{ in.}$$

A minimum plate thickness of 1.25 inches will be used.

Determine the Support Configuration and Check Bearing Stress

Superstructure support is provided by 12-inch-wide pile caps on existing abutments. The bridge will be anchored to the caps with bolts placed through the deck and cap:



Bearing stress is checked for the bearing length, ℓ_b, of 12 inches. From Table 16-8, the maximum reaction for one wheel line of an HS 20-44 truck on a 37-foot span is 26,920 pounds. The dead load reaction is computed using the bridge length of 38 feet:

$$R_{DL} = \frac{w_{DL}(38 \text{ ft})}{2} = \frac{(585.4 \text{ lb/ft})(38 \text{ ft})}{2} = 11{,}123 \text{ lb}$$

Bearing stress in compression perpendicular to grain is computed by Equation 9-32:

$$f_{c\perp} = \frac{R_{DL} + R_{LL}}{D_W(\ell_b)} = \frac{11{,}123 + 26{,}920}{63(12)} = 50.3 \text{ lb/in}^2$$

$f_{c\perp} = 50.3 \text{ lb/in}^2 < F_{c\perp} = 419 \text{ lb/in}^2$, so the bearing configuration is satisfactory.

Summary

The replacement bridge will consist of a longitudinal stress-laminated lumber deck, 38 feet long, with a span of 37 feet center-to-center of bearings. The bridge will be 26 feet wide and carry two lanes of AASHTO HS 20-44 loading on a roadway width of 24 feet. The lumber laminations will be S1S full-sawn 4-inch by 16-inch Douglas Fir-Larch, visually graded No. 1 or better. The stressing system will consist of galvanized 1-inch-diameter high-strength steel rods conforming to ASTM A722. The rods will be spaced 48 inches on center with end rods 12 inches from the deck end. The rod anchorage system will consist of a 14-inch by 14-inch by 1.25-inch bearing plate and a 4-inch by 6.5-inch by 1.25-inch anchorage plate, manufactured of galvanized A36 steel.

Stresses, deflections, prestressing force and camber are as follows:

$$f_b = 1{,}666 \text{ lb/in}^2$$

$$F_b' = 1{,}935 \text{ lb/in}^2$$

$$\Delta_{LL} = 1.19 \text{ in.} = L/373$$

$$\Delta_{DL} = 0.82 \text{ in.}$$

$$\text{Camber} = 2.5 \text{ in.}$$

$$N = 40 \text{ lb/in}^2$$

$$N_i = 100 \text{ lb/in}^2$$

$$F_{ps} = 76{,}800 \text{ lb}$$

$$f_{c\perp} \text{ at anchorage} = 392 \text{ lb/in}^2$$

$$f_{c\perp} \text{ at bearings} = 50.3 \text{ lb/in}^2$$

$$F_{c\perp}' = 419 \text{ lb/in}^2$$

Example 9.2 - Channel bulkhead anchorage for longitudinal stress-laminated lumber decks

Design a channel bulkhead anchorage for the bridge of Example 9-1. The following values apply:

$$F_{ps} = 76{,}800 \text{ lb}$$

$$F_{c\perp}' = 419 \text{ lb/in}^2$$

Solution

The channel bulkhead configuration is illustrated in Figure 9-20. Design will involve selecting a channel size and a bearing plate size, then checking bearing stress on the lumber laminations.

Determine Channel Size

By Equation 9-25, the channel depth must be within 85 to 100 percent of the deck thickness:

$$0.85(t) = 0.85(16) = 13.4 \text{ in.}$$

$$13.4 \text{ in.} \leq d_c \leq 16 \text{ in.}$$

From Table 9-7, minimum channel section properties for a 16-inch deck are as follows:

$I = 9.2 \text{ in}^4$

$t_w = 0.52 \text{ in.}$

From the *Steel Construction Manual,*[3] a C15x40 channel is selected with the following properties:

$d_c = 15$ in.

$I = 9.23 \text{ in}^4$

$t_w = 0.52 \text{ in.}$

Determine Bearing Plate Size

The minimum bearing plate length is computed by Equation 9-26:

$$L_p \geq \frac{F_{ps}}{d_c F_{c\perp}'} - 2(t_w) = \frac{76{,}800}{15(419)} - 2(0.52) = 11.2 \text{ in.}$$

An initial plate size of 12 inches by 10 inches is selected and the length/width ratio is checked by Equation 9-27:

$L_p = 12$in.

$W_p = 10$ in.

$$\frac{L_p}{W_p} = \frac{12}{10} = 1.2$$

The ratio is between 1.0 and 2.0 and is acceptable.

Minimum plate thickness is computed by Equation 9-28:

$$t_p \geq \frac{L_p}{12} = \frac{12}{12} = 1 \text{ in.}$$

A plate thickness of 1 inch will be used.

Check Bearing Stress

The effective bearing area of the channel bulkhead is computed by Equation 9-29:

$$A_g = d_c(L_p + 2t_w) = 15[12 + (2)(0.52)] = 196 \text{ in}^2$$

Bearing stress is computed by Equation 9-30:

$$f_{c\perp} = \frac{F_{ps}}{A_g} = \frac{76{,}800}{196} = 392 \text{ lb/in}^2$$

$f_{c\perp} = 392$ lb/in² $< F_{c\perp}' = 419$ lb/in², so the channel and plate sizes are acceptable.

Summary

The anchorage will consist of a C15x40 steel channel with 12-inch by 10-inch by 1-inch bearing plates.

Example 9-3 - Longitudinal stress-laminated lumber deck; single lane, HS 25-44 loading

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A single-lane stress-laminated lumber bridge will be built on a remote logging road where the design speed is 5 mph. The bridge will span 22 feet center-to-center of bearings and carry one lane of AASHTO HS 25-44 loading a roadway width of 14 feet. Bridge ends are supported on abutments with a 12-inch length of bearing. Design this bridge, assuming the following:

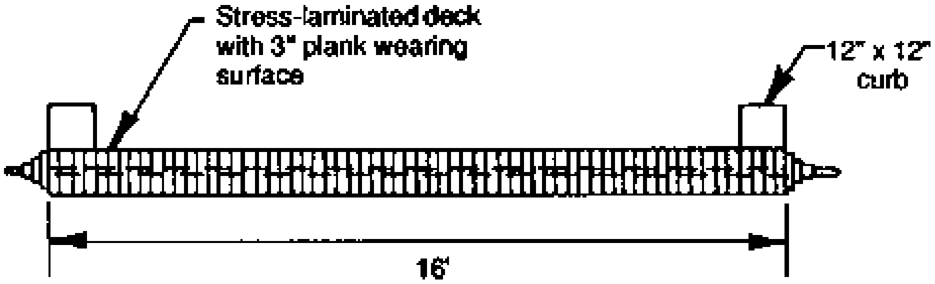
1. The bridge will include 12-inch by 12-inch timber curbs and a 3-inch thick lumber wearing surface.

2. Because of the low design speed, live load deflection is not a consideration.

3. Lumber laminations are surfaced Red Pine.

4. Butt joints are not required.

5. A bearing plate anchorage configuration will be used.



Solution

Define Deck Geometric Requirements and Design Loads

The bridge supports one traffic lane over a 22-foot span. With a roadway width of 14 feet and 12-inch-wide curbs, a bridge width of 16 feet is required. Design loading will be one HS 25-44 wheel line in AASHTO Load Group I.

Select a Species and Grade of Lamination and Compute Allowable Design Values

From the NDS Table 4A, No. 1 Red Pine visually graded to NLGA rules is selected. Tabulated values are as follows:

F_b = 1,000 lb/in^2 (single-member use)

E = 1,300,000 lb/in^2

$F_{c\perp}$ = 440 lb/in^2

Allowable design values are computed using the applicable moisture content factor (C_M) from Table 5-7:

$$F_b' = F_b C_M C_{LS} = 1{,}000(0.86)(1.50) = 1{,}290 \text{ lb/in}^2$$

$$E' = EC_M = 1{,}300{,}000(0.97) = 1{,}261{,}000 \text{ lb/in}^2$$

$$F_{c\perp}' = F_{c\perp} C_M = 440(0.67) = 294 \text{ lb/in}^2$$

Determine the Preliminary Lamination Layout

Lumber laminations will be continuous over the bridge span. Butt joints. are not required.

Compute the Transverse Moduli for the Stress-Laminated System

Values of the transverse bending modulus (E_{TS}) and transverse shear modulus (G_{TS}) are computed by Equations 9-4 and 9-5:

$$E_{TS} = 0.013E' = 0.013(1{,}261{,}000) = 16{,}393 \text{ lb/in}^2$$

$$G_{TS} = 0.03E' = 0.03(1{,}261{,}000) = 37{,}830 \text{ lb/in}^2$$

Compute Maximum Live Load Moment

The maximum live load moment for one wheel line of an HS 25-44 truck on a 22-foot simple span is obtained from Table 16-8:

$$M_{LL} = 110{,}000 \text{ ft-lb}$$

Compute Wheel Load Distribution Width

Values of α and θ are computed using Equations 9-6 and 9-7, respectively. From Table 9-1, $C_B = 1.0$:

$$\alpha = \frac{2G_{TS}}{\sqrt{E'(C_B)(E_{TS})}} = \frac{2(37{,}830)}{\sqrt{1{,}261{,}000(1.0)(16{,}393)}} = 0.53$$

$$\theta = \frac{b}{2L}\left[\frac{E'(C_B)}{E_{TS}}\right]^{0.25} = \frac{16}{2(22)}\left[\frac{1{,}261{,}000(1.0)}{16{,}393}\right]^{0.25} = 1.08$$

The distribution width, D_w, is obtained from Figure 9-15 using the curves for bridges with one traffic lane:

$$D_W = 62 \text{ in.}$$

Estimate Deck Thickness and Compute Effective Section Properties

An initial deck thickness of 11-1/4 inches (12 inches nominal) is selected from Table 9-2. Although the table is based on HS 20-44 loading, for this span it should be reasonably accurate for HS 25-44 loads. Effective deck section properties are computed by Equations 9-8 and 9-9:

$$S = \frac{D_W(C_B)(t)^2}{6} = \frac{62(1.0)(11.25)^2}{6} = 1{,}308 \text{ in}^3$$

$$I = \frac{D_W(C_B)(t)^3}{12} = \frac{62(1.0)(11.25)^3}{12} = 7{,}356 \text{ in}^4$$

Compute Deck Dead Load and Dead Load Moment

From Table 9-3, the dead load of an 11.25inch deck with a 3-inch lumber wearing surface is 59.4 lb/ft^2 Based on a unit weight for wood of 50 lb/ft^3, curb dead load is 50 lb/ft. The curb dead load is increased by an estimated 10 lb/ft for the prestressing system, and is assumed to be uniformly distributed across the deck width:

$$\text{DL} = 59.4 \text{ lb/ft}^2 + \frac{2(50 \text{ lb/ft} + 10 \text{ lb/ft})}{16 \text{ ft}} = 66.9 \text{ lb/ft}^2$$

For the distribution width of 62 in.,

$$w_{DL} = \frac{62 \text{ in.}}{12 \text{ in/ft}}\left(66.9 \text{ lb/ft}^2\right) = 345.7 \text{ lb/ft}$$

$$M_{DL} = \frac{w_{DL}L^2}{8} = \frac{345.7(22)^2}{8} = 20{,}915 \text{ ft-lb}$$

Compute Bending Stress

Bending stress is computed by Equation 9-11:

$$f_b = \frac{M}{S} = \frac{(110{,}000 + 20{,}915)(12 \text{ in/ft})}{1{,}308} = 1{,}201 \text{ lb/in}^2$$

f_b = 1,201 lb/in^2 < F_b' = 1,290 lb/in^2, so bending stress is acceptable.

Check Live Load Deflection

Although live load deflection is not a controlling consideration for design, it will be computed for reference. From Table 16-8, the deflection coefficient for one wheel line of an HS 25-44 truck on a 22-foot simple span is 7.99 x 10^9 lb-in^2. Live load deflection is computed using 133 percent of the effective deck moment of inertia:

$$\Delta_{LL} = \frac{7.99 \times 10^9}{E'(1.33)(I)} = \frac{7.99 \times 10^9}{1{,}261{,}000(1.33)(7{,}356)} = 0.65 \text{ in.}$$

Determine the Required Prestress Level

Using the previously computed values of α and θ, M_T is obtained for HS 20-44 loading on a two-lane bridge from Figure 9-16:

HS 20-44 M_T = 480 in-lb/in

Because this design is for HS 25-44 loading, the value of M_T from Figure 9-16 must be multiplied by the ratio of the design wheel load (20,000 pounds from Example 6-1) to the HS 20-44 wheel load (16,000 pounds):

$$\frac{20{,}000 \text{ lb}}{16{,}000 \text{ lb}} = 1.25$$

M_T = 1.25 (480 in-lb/in) = 600 in-lb/in

The variable β is computed by Equation 9-14:

$$\beta = \pi\left(\frac{b}{L}\right)\sqrt{\frac{E'(C_s)}{2G_{TS}}} = 3.14\left(\frac{16}{22}\right)\sqrt{\frac{1{,}261{,}000(1.0)}{2(37{,}830)}} = 9.32$$

By interpolation and extrapolation of Figure 9-17,

HS 20-44 V_T = 60 lb/in

For HS 25-44 loading,

$$V_T = 1.25(60 \text{ lb/in}) = 75 \text{ lb/in.}$$

The minimum level of compressive prestress is computed is computed by Equation 9-15. Based on transverse bending,

$$N = \frac{6M_T}{t^2} = \frac{6(600)}{(11.25)^2} = 28.4 \text{ lb/in}^2$$

Based on transverse shear,

$$N = \frac{1.5V_T}{t\mu} = \frac{1.5(75)}{11.25(0.35)} = 28.6 \text{ lb/in}^2$$

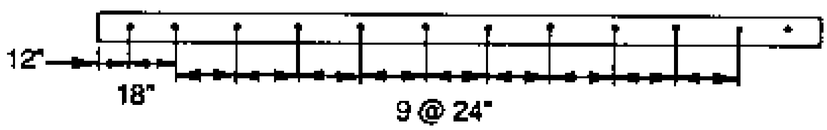
Both values are less than the minimum 40 lb/in^2, so N = 40 lb/in^2 will control. By Equation 9-17,

$$N_i = 2.5N = 2.5(40) = 100 \text{ lb/in}^2$$

Determine Spacing and Size of Prestressing Rods and the Required Prestressing Force

From Table 9-5 for an 11.25-inch deck, two rod diameters are feasible; 5/8-inch-diameter rods at a spacing of 16 to 26 inches, or 1-inch-diameter rods at a spacing of 47 to 79 inches. From Figure 9-18, maximum rod spacing is limited to approximately 58 inches.

It is anticipated that 1-inch-diameter rods at the minimum 47-inch spacing will require an excessive bearing plate size. Therefore, 5/8-inch-diameter rods will be used. For a bridge length of 23 feet (22- foot span on 1-foot-wide sills), rods will be spaced 24 inches on-center with the end rods spaced at 12 inches and 18 inches:



From Table 9-4 for a 5/8-inch-diameter rod, A_S = 0.28 in^2. The minimum required rod area and the steel/wood ratio are checked by Equation 9-18:

$$\frac{N_i S_p t}{0.70 f_{pu}} = \frac{100(24)(11.25)}{0.70(150{,}000)} = 0.26 \text{ in}^2 < 0.28 \text{ in}^2$$

$$\frac{A_S}{S_P(t)} = \frac{0.28}{24(11.25)} = 0.0010 < 0.0016$$

The prestressing force required in each rod, F_{ps} is computed by Equation 9-19:

$$F_{ps} = N_i(S_p)(t) = 100(24)(11.25) = 27{,}000 \text{ lb}$$

Design Anchorage System

The minimum bearing plate area is computed by Equation 9-20:

$$A_p = \frac{F_{ps}}{F_{c\perp}'} = \frac{27{,}000}{294} = 92 \text{ in}^2$$

For the 11.25-inch-thick deck, a plate depth, W_p, of 10 inches is chosen. The minimum required plate length is computed by dividing the plate area by the plate width:

$$L_p = \frac{A_p}{W_p} = \frac{92}{10} = 9.2 \text{ in.}$$

A 10-inch-square plate will be used, and

W_p = 10 in.

L_p = 10 in.

A_p = 100 in

For the square plate, the ratio of the bearing plate length to width is acceptable by Equation 9-21, and bearing stress in compression perpendicular to grain is computed by Equation 9-22:

$$f_{c\perp} = \frac{F_{ps}}{A_p} = \frac{27{,}000}{100} = 270 \text{ lb/in}^2$$

$f_{c\perp} = 270 \text{ lb/in}^2 < F_{c\perp}' = 294 \text{ lb/in}^2$, so bearing stress is acceptable.

From Table 9-6, an anchorage plate size of 3 inches by 3 inches by 0.75 inch is selected and *k* values are computed by Equation 9-24:

W_A = 3 in.

L_A = 3 in.

t_A = 0.75 in.

$$k = \frac{W_P - W_A}{2} = \frac{L_P - L_A}{2} = \frac{10 - 3}{2} = 3.5 \text{ in.}$$

The required bearing plate thickness for an A36 steel plate is computed by Equation 9-23:

$$t_p = \sqrt{\frac{3(f_{c\perp})(k^2)}{F_b}} = \sqrt{\frac{3(270)(3.5)^2}{0.55(36{,}000)}} = 0.71 \text{ in.}$$

A plate thickness of 0.75 inch will be used.

Determine the Support Configuration and Check Bearing Stress

Superstructure support is provided by a bearing length, ℓ_b, of 12 inches. From Table 16-8, the maximum reaction for one wheel line of an HS 25-44 truck on a 22-foot span is 27,270 pounds. The dead load reaction is computed using the bridge length of 23 feet:

$$R_{DL} = \frac{w_{DL}(23 \text{ ft})}{2} = \frac{(345.7 \text{ lb/ft})(23 \text{ ft})}{2} = 3{,}976 \text{ lb}$$

Bearing stress in compression perpendicular to gram is computed by Equation 9-32:

$$f_{c\perp} = \frac{R_{DL} + R_{LL}}{D_W(\ell_b)} = \frac{3{,}976 + 27{,}270}{62(12)} = 42 \text{ lb/in}^2$$

$f_{c\perp} = 42$ lb/in^2 $< F_{c\perp}' =$ 294 lb/in^2, so the bearing configuration is satisfactory.

Summary

The bridge will consist of a longitudinal stress-laminated lumber deck, 23 feet long, with a span of 22 feet center to center of bearings. The bridge will be 16 feet wide and carry one lane of AASHTO HS 25-44 loading on a roadway width of 14 feet. The lumber laminations will be S4S 2-inch by 12-inch Red Pine, visually graded No. 1 or better to NLGA rules. The stressing system will consist of galvanized 5/8-inch-diameter high-strength steel rods conforming to ASTM A722, spaced 24 inches on-center. The rod anchorage system will consist of a 10-inch by 10-inch by 0.75-inch bearing plate and a 3-inch by 3-inch by 0.75-inch anchorage plate, manufactured of galvanized A36 steel.

Stresses, deflections, prestressing force and camber are as follows:

$f_b =$ 1,201 lb/in^2

$F_b' =$ 1,290 lb/in^2

$$\Delta_{LL} = 0.65 \text{ in.}$$

$$N = 40 \text{ lb/in}^2$$

$$N_i = 100 \text{ lb/in}^2$$

$$F_{ps} = 27{,}000 \text{ lb}$$

$$f_{c\perp} \text{ at anchorage} = 270 \text{ lb/in}^2$$

$$f_{c\perp} \text{ at bearings} = 42 \text{ lb/in}^2$$

$$F_{c\perp}' = 294 \text{ lb/in}^2$$

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